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AISI Manual -- Cold-Formed Steel Design, 2008 Edition

American Iron and Steel Institute

AISI MANUAL

Cold-Formed Steel Design

2008 Edition

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SECTION GUIDE

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The material contained herein has been developed by the American Iron and Steel Institute Committee on Specifications for the Design of Cold-Formed Steel Structural Members. The Committee has made a diligent effort to present accurate, reliable, and useful information on cold-formed steel design. The Committee acknowledges and is grateful for the contributions of the numerous researchers, engineers, and others who have contributed to the body of knowledge on the subject.

With anticipated improvements in understanding of the behavior of cold-formed steel and the continuing development of new technology, this material may eventually become dated. It is anticipated that AISI will publish updates of this material as new information becomes available, but this cannot be guaranteed.

The materials set forth herein are for general information only. They are not a substitute for competent professional advice. Application of this information to a specific project should be reviewed by a registered professional engineer. Indeed, in most jurisdictions, such review is required by law. Anyone making use of the information set forth herein does so at their own risk and assumes any and all resulting liability arising therefrom.

AISI *Cold-Formed Steel Design Manual* has been produced by the Steel Market Development Institute, a business unit of AISI.

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Produced by Computerized Structural Design, S.C.
Milwaukee, Wisconsin

PREFACE

The 2008 edition of the *Cold-Formed Steel Design Manual* consists of six Parts. This information is supplemental to the 2007 edition of the *North American Specification for the Design of Cold-Formed Steel Structural Member*s. Each part in the *Design Manual* should be used in conjunction with the *Specification, Commentary* and the other parts, where appropriate.

Part I, Dimensions and Properties contains (a) information regarding the availability and properties of steels referenced in the *Specificatio*n, (b) tables of section properties, and (c) formulas and examples of calculations of section properties.

Part II, Beam Design contains (a) tables and charts to aid in beam design, and (b) beam design example problems.

Part III, Column Design contains (a) tables to aid in column design, and (b) column design example problems.

Part IV, Connections contains (a) tables to aid in connection design, and (b) connection example problems.

Part V, Supplementary Information contains (a) a table of *Specification* cross-references to the examples provided in this manual (b) design procedures of specification nature which are not included in the *Specification* itself, either because they are infrequently used or are regarded as too complex for routine design, and (c) other information intended to assist users of cold-formed steel.

Part VI, Test Standards contains (a) test methods for cold-formed steel, (b) a bibliography of other pertinent test methods, and (c) an example problem.

In addition to updating the *Design Manual* for conformance with the 2007 edition of the *North American Specification*, the following improvements or additions have been made:

- A series of brief discussions is provided in Parts I-IV, which summarize the intent of the *Specification* provisions and provide design guideline for users.
- Standard studs and tracks produced by members of the Steel Stud Manufacturers Association (SSMA) are included in Parts I through IV.
- Formulas for calculating section properties used in distortional buckling analysis are added, along with distortional buckling tables for C-, Z- and SSMA stud sections.
- Tables for screw shear, pull-out, and pull-over are provided specifically for SSMA sections.
- A table is added for arc spot welds for the case of shear of a sheet welded to an identical sheet.
- The following new design examples have been added to illustrate new design provisions in the *Specification*:
  (a) Round and rectangular tubular section member design in Parts II and III,
  (b) C-section member subjected to combined bending and torsional loading in Part II,
  (c) Distortional buckling of C-Section members in Parts II and III,
  (d) Sigma-shaped flexural and compression member design by the Direct Strength Method in Parts II and III,

(e) Web crippling strength of beam webs with bearing stiffeners in Part II, and
(f) Frame Design with consideration of second order analysis in Part III.

- The Direct Strength Method is introduced for designing members subject to local, distortional and/or global buckling.
- A table of cross-references between the new and previous Test Standard designations is provided in Part VI.
- Editorial changes have been made to all the test standards. Technical guidance and background information have been added to the standards as user notes and commentary. The following six new test standards are included in Part VI:
  1. AISI S909-08, *Standard Test Method for Determining the Web Crippling Strength of Cold-Formed Steel Beams*,
  2. AISI S910-08, *Test Method for Distortional Buckling of Cold-Formed Steel Hat Shaped Compression Members*,
  3. AISI S911-08, *Method for Flexural Testing Cold-Formed Steel Hat Shaped Beams*,
  4. AISI S912-08, *Test Procedure for Determining a Strength Value for a Roof Panel-To-Purlin-To-Anchorage Device Connection*,
  5. AISI S913-08, *Test Standard for Hold-Downs Attached to Cold-Formed Steel Structural Framing*, and
  6. AISI S914-08, *Test Standard For Joist Connectors Attached to Cold-Formed Steel Structural Framing*.

AISI acknowledges the technical information provided by the Steel Deck Institute in the Steel Deck section Part I, the section geometries provided by the Steel Stud Manufacturers Association in Part I, and the exemplary efforts of Richard C. Kaehler with Computerized Structural Design, S. C., in developing this *Design Manual*. Special thanks also are also extended to the members of the AISI Design Manual Subcommittee:

Paul A. Seaburg, *Chairman*	Don Allen	Reidar Bjorhovde
James K. Crews	Daniel A. Cuoco	Cynthia J. Duncan
Robert S. Glauz	Richard B. Haws	Donald L. Johnson
Richard C. Kaehler	Roger A. LaBoube	Thomas M. Murray
Joe N. Nunnery	Benjamin W. Schafer	Karl Schroeder
Reinhold M. Schuster	Wei-Wen Yu	

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PART I
DIMENSIONS AND PROPERTIES

For Use With the 2007 Edition of the North American Specification for the Design of Cold-Formed Steel Structural Members

PART I - DIMENSIONS AND PROPERTIES

SECTION 1 - STEELS - AVAILABILITY AND PROPERTIES

1.1 Introduction To Table Of Referenced Steels

The table of referenced steels is provided as a guide for material selection. It summarizes the scope of the corresponding ASTM Standards, identifies product classifications and lists important material properties.

Referenced Steels

There are 16 referenced ASTM Standards for steels that are accepted for use with the *North American Specification for the Design of Cold-Formed Steel Structural Members (Specification)*. Use of these referenced steels is encouraged; however, other steels may also be used in cold-formed steel structures provided they satisfy the provisions of *Specification* Section A2.2.

Product Classifications

Of the referenced steels, 5 are for plate and bar, 1 is for plate, 2 are for sheet and strip, 6 are for sheet, and 2 are for tubing products.

ASTM classifies hot-rolled steel products as being either sheet, strip, plate or bar, based on size, as follows:

Product Classification - Hot-Rolled Steel

Width, w in.	Thickness, t in.			
	0.2300 ≤ t	0.203 ≤ t ≤ 0.230	0.1800 ≤ t ≤ 0.2030	0.044 ≤ t ≤ 0.180
w ≤ 3-1/2	bar	bar	strip	strip
3-1/2 < w ≤ 6	bar	bar	strip	strip
6 < w ≤ 8	bar	strip	strip	strip
8 < w ≤ 12	plate (1)	strip	strip	strip
12 < w ≤ 48	plate (2)	sheet	sheet	sheet
48 < w	plate (2)	plate (2)	plate (2)	sheet

(1) Strip, only when ordered in coils.
(2) Sheet, only when ordered in coils.

ASTM classifies cold-rolled carbon and high-strength low-alloy (HSLA) sheet steel products, including hot-dip coated, based on size, as follows:

Product Classification - Cold-Rolled Sheet Steel

Width, w in.	Thickness, t in.	
	Carbon Steel	HSLA steel
w ≤ 12	t ≤ 0.082	0.019 ≤ t ≤ 0.082
12 < w	t ≤ 0.142	0.020 ≤ t

Structural Properties

The structural properties significant to cold-formed steel structures are listed in the table of referenced steels, and include yield stress, tensile strength, elongation in 2 inches, and the ratio of tensile strength to yield stress. Total elongation in 2 inches is a measure of ductility, the ability of a steel to undergo sizable plastic or permanent strains before fracturing. The ratio of tensile strength to yield stress is an indication of the ability of the material to redistribute stress.

1.2 Summary Of Scope And Principle Tensile Properties, ASTM Specifications For Referenced Steels

Table 1.2

Summary Of Scope And Principle Tensile Properties
ASTM Specifications for Referenced Steels

ASTM Designation SCOPE (After ASTM)	PRODUCT	GRADE	F_y ksi (min)	F_u ksi (min/max)	Percent elongation in 2 inches (min)	$\frac{F_{u(min)}}{F_{y(min)}}$
A36/A36M-05 This specification covers carbon steel shapes, plates and bars for use in riveted, bolted, or welded construction of bridges and buildings, and for general structural purposes. Supplemental requirements are provided where improved notch toughness is important. These apply only when specified by the purchaser in the order. When the steel is to be welded, a welding procedure suitable for the grade of steel and intended use or service is to be utilized.	Plates and Bars	--	36	58 / 80	23	1.61
A242/A242M-04e1 This specification covers high-strength low-alloy structural steel shapes, plates and bars for welded, riveted, or bolted construction intended primarily for use as structural members where savings in weight or added durability are important. The atmospheric corrosion resistance of the steel in most environments is substantially better than that of carbon structural steels with or without copper addition. When properly exposed to the atmosphere, this steel can be used bare (unpainted) for many applications. This specification is limited to material up to 4 in. [100 mm], inclusive, in thickness. When the steel is to be welded, it is presupposed that a welding procedure suitable for the grade of steel and intended use or service is to be utilized.	Plates and Bars $t \leq 3/4$ in.	--	50	70	21	1.40
A283/A283M-03 This specification covers four grades of carbon steel plates for general applications. When the steel is to be welded, a welding procedure suitable for the grade of steel and intended use or service is to be utilized.	Plate	A B C D	24 27 30 33	45 / 60 50 / 65 55 / 75 60 / 80	30 28 25 23	1.88 1.85 1.83 1.82

Table 1.2

Summary Of Scope And Principle Tensile Properties
ASTM Specifications for Referenced Steels

ASTM Designation SCOPE (After ASTM)	PRODUCT	GRADE	F_y ksi (min)	F_u ksi (min/max)	Percent elongation in 2 inches (min)	$\frac{F_{u(min)}}{F_{y(min)}}$
A500-03a This specification covers cold-formed welded and seamless carbon steel round, square, rectangular, or special shape structural tubing for welded, riveted, or bolted construction of bridges and buildings, and for general structural purposes. This tubing is produced in both welded and seamless sizes with a maximum periphery of 64 in. [1626 mm] and a maximum wall of 0.625 in. [15.88 mm]. Grade D requires heat treatments. *Note:* Products manufactured to this specification may not be suitable for those applications such as dynamically loaded elements in welded structures, etc., where low-temperature notch-toughness properties may be important.	Round Tubing Shaped Tubing	A B C D A B C D	33 42 46 36 39 46 50 36	45 58 62 58 45 58 62 58	25 23 21 23 25 23 21 23	1.36 1.38 1.35 1.61 1.15 1.26 1.24 1.61
A529/A529M-05 This specification covers carbon-manganese steel shapes, plates and bars for use in riveted, bolted, or welded construction of buildings and for general structural purposes. Material under this specification is available in two grades: Grade 50 for plates to 1 in. [25.4 mm] thick and to 15 in. [380 mm] wide, bars to 3 1/2 in. [90 mm] thick, and shapes with flange or leg thickness to 1 ½ in. [40 mm]; Grade 55 for plates to 1 in. [25.4 mm] thick and to 15 in. [380 mm] wide, bars to 3 in. [75 mm] thick, and shapes with flange or leg thickness to 1 ½ in. [40 mm]. When the steel is to be welded, a welding procedure suitable for the grade of steel and intended use or service is to be utilized.	Plates and Bars	50 55	50 55	70 / 100 70 / 100	21 20	1.40 1.27
A572/A572M-06 This specification covers five grades of high-strength low-alloy structural steel shapes, plates, sheet piling, and bars. Grades 42 [290], 50 [345], and 55 [380] are intended for riveted, bolted, or welded construction. Grades 60 [415] and 65 [450] are intended for riveted or bolted construction of bridges or for riveted, bolted, or welded construction in other applications. For applications such as welded bridge construction, where notch toughness is important, notch toughness requirements are to be negotiated between the purchaser and the producer. When the steel is to be welded, a welding procedure suitable for the grade of steel and intended use or service is to be utilized.	Plates and Bars	42 50 55 60 65	42 50 55 60 65	60 65 70 75 80	24 21 20 18 17	1.43 1.30 1.27 1.25 1.23

Table 1.2

Summary Of Scope And Principle Tensile Properties
ASTM Specifications for Referenced Steels

ASTM Designation SCOPE (After ASTM)	PRODUCT	GRADE	F_y ksi (min)	F_u ksi (min/max)	Percent elongation in 2 inches (min)	$\frac{F_{u(min)}}{F_{y(min)}}$
A588/A588M-05 This specification covers high-strength low-alloy structural steel shapes, plates and bars for welded, riveted, or bolted construction but intended primarily for use in welded bridges and buildings where savings in weight or added durability are important. The atmospheric corrosion resistance of this steel in most environments is substantially better than that of carbon structural steels with or without copper addition. When properly exposed to the atmosphere this steel can be used bare (unpainted) for many applications. This specification is limited to material up to 8 in. [200 mm] inclusive in thickness. When the steel is to be welded, a welding procedure suitable for the grade of steel and intended use or service is to be utilized.	Plates and Bars $t \leq 4$ in.	--	50	70	21	1.40
A606-04 This specification covers high-strength, low-alloy, hot- and cold-rolled sheet and strip in cut lengths or coils, intended for use in structural and miscellaneous purposes, where savings in weight or added durability are important. These steels have enhanced atmospheric corrosion resistance and are supplied in two types: Type 2 contains 0.20% minimum copper based on cast or heat analysis (0.18% minimum Cu for product check). Type 4 contains additional alloying elements and provides a level of corrosion resistance substantially better than that of carbon steels with or without copper addition. When properly exposed to the atmosphere, Type 4 can be used bare (unpainted) for many applications.	Sheet and Strip	Hot Rolled -As Rolled Hot Rolled -Annealed or Normalized Cold Rolled	50 45 45	70 65 65	22 22 22	1.40 1.44 1.44

Table 1.2

Summary Of Scope And Principle Tensile Properties
ASTM Specifications for Referenced Steels

ASTM Designation SCOPE (After ASTM)	PRODUCT	GRADE	F_y ksi (min)	F_u ksi (min/max)	Percent elongation in 2 inches (min)	$\frac{F_{u(min)}}{F_{y(min)}}$
A653/A653M-06 This specification covers steel sheet, zinc coated (galvanized) or zinc-iron alloy coated (galvannealed) by the hot dip process in coils and cut lengths. Included are several grades based on yield stress in both structural steel (SS), high-strength low-alloy (HSLAS), and high-strength low-alloy with improved formability (HSLA-F). Products furnished under A653/A653M must conform to the latest revision of A924/A924M except as otherwise indicated in the specification.	Sheet	SS				
		33	33	45	20	1.36
		37	37	52	18	1.41
		40	40	55	16	1.38
		50 Class 1	50	65	12	1.30
		50 Class 3	50	70	12	1.40
		50 Class 4	50	60	12	1.20
		55	55	70	11	1.27
		80	80	82	-	1.03
		HSLAS				
		40	40	50	22	1.25
		50	50	60	20	1.20
		55 Class 1	55	70	16	1.27
		55 Class 2	55	65	18	1.18
		60	60	70	16	1.17
		70	70	80	12	1.14
		80	80	90	10	1.13
		HSLAS-F				
		40	40	50	24	1.25
		50	50	60	22	1.20
		55 Class 1	55	70	18	1.27
		55 Class 2	55	65	20	1.18
		60	60	70	18	1.17
		70	70	80	14	1.14
		80	80	90	12	1.13
A792/A792M-05 This specification covers 55% aluminum-zinc alloy-coated steel sheet in coils and cut lengths The product is intended for applications requiring corrosion resistance or heat resistance or both. The product is available as Commercial Steel (CS), Forming Steel (FS), Drawing Steel (DS), High Temperature Steel (HTS), and Structural Steel (SS).	Sheet	SS				
		33	33	45	20	1.36
		37	37	52	18	1.41
		40	40	55	16	1.38
		50 Class 1	50	65	12	1.30
		50 Class 4	50	60	12	1.20
		80	80	82	-	1.03

Table 1.2

Summary Of Scope And Principle Tensile Properties
ASTM Specifications for Referenced Steels

ASTM Designation SCOPE (After ASTM)	PRODUCT	GRADE	F_y ksi (min)	F_u ksi (min/max)	Percent elongation in 2 inches (min)	$\frac{F_{u(min)}}{F_{y(min)}}$
A847-05 This specification covers cold-formed welded and seamless high-strength low-alloy round, square, rectangular, or special shaped structural tubing for welded, riveted, or bolted construction of bridges and buildings and for general structural purposes where high strength and enhanced atmospheric corrosion resistance are required. The atmospheric corrosion resistance of this steel in most environments is substantially better than carbon steel with or without copper addition. When properly exposed to the atmosphere, this steel can be used bare (unpainted) for many applications. When this steel is used in the welded construction, the welding procedure shall be suitable for the steel and the intended service. This tubing is produced in welded sizes with a maximum periphery of 64 in. [1626 mm] and a maximum wall of 0.625 in. [15.88 mm], and in seamless with a maximum periphery of 32 in. [813 mm] and a maximum wall of 0.500 in. [12.70 mm].	Round and Shaped Tubing	-	50	70	19	1.40
A875/A875M-99 This specification covers steel sheet, in coils and cut lengths, metallic-coated by the hot-dip process, with zinc-5% aluminum alloy coating. The Zn-5Al alloy coating also contains small amounts of elements other than zinc and aluminum that are intended to improve processing and the characteristics of the coated product. The coating is produced as two types: zinc-5% aluminum-mischmetal alloy (Type I) and zinc-5% aluminum-0.1% magnesium alloy (Type II), and in two coating structures (classes). The coated sheet is produced in several coating designations (coating weight [mass]). The material is intended for applications requiring corrosion resistance, formability, and paintability. The steel sheet is produced in a number of designations, types, grades, and classes designed to be compatible with differing application requirements.	Sheet	SS				
		33	33	45	20	1.36
		37	37	52	18	1.41
		40	40	55	16	1.38
		50 Class 1	50	65	12	1.30
		50 Class 3	50	70	12	1.40
		80	80	82	--	--
		HSLAS				
		50	50	60	20	1.20
		60	60	70	16	1.17
		70	70	80	12	1.14
		80	80	90	10	1.12
		HSLAS-F				
		50	50	60	22	1.20
		60	60	70	18	1.17
		70	70	80	14	1.14
		80	80	90	12	1.12

Table 1.2

Summary Of Scope And Principle Tensile Properties
ASTM Specifications for Referenced Steels

ASTM Designation SCOPE (After ASTM)	PRODUCT	GRADE	F_y ksi (min)	F_u ksi (min/max)	Percent elongation in 2 inches (min)	$\frac{F_{u(min)}}{F_{y(min)}}$
A1003/A1003M-05						
This specification covers coated steel sheet used in the manufacture of cold-formed framing members, such as, but not limited to, studs, joists, purlins, girts, and track. The sheet steel used for cold-formed framing members includes metallic coated, painted metallic coated, or painted nonmetallic-coated. The grade designations use the following suffix indicators: H - high ductility, L - low ductility, and NS - nonstructural. H and L are associated with structural or load-bearing applications, and NS with nonstructural or nonload-bearing applications.	Sheet	ST33H	33	45	10	1.36*
		ST37H	37	52	10	1.40*
		ST40H	40	55	10	1.37*
		ST50H	50	66	10	1.32*
		ST33L	33	-	3	-
		ST37L	37	-	3	-
		ST40L	40	-	3	-
		ST50L	50	-	3	-
*Additionally, test values must show a minimum value of 1.08 for F_u/F_y.						

Table 1.2

Summary Of Scope And Principle Tensile Properties
ASTM Specifications for Referenced Steels

ASTM Designation SCOPE (After ASTM)	PRODUCT	GRADE	F_y ksi (min)	F_u ksi (min/max)	Percent elongation in 2 inches (min)	$\frac{F_{u(min)}}{F_{y(min)}}$
A1008/A1008M-05b This specification covers cold-rolled structural, high-strength low-alloy, and high-strength low-alloy with improved formability, solution hardened, and bake hardenable steel sheet, in coils and cut lengths. The product is produced in a number of designations including structural steel (SS), high-strength low-alloy steel (HSLAS), and high-strength low-alloy steel with improved formability (HSLAS-F). HSLAS-F steel has improved formability compared to HSLAS. The steel is fully deoxidized, made to fine grain practice, and includes microalloying elements such as columbium, vanadium, and zirconium. The steel may be treated to achieve inclusion control. Cold-rolled steel sheet is supplied for either exposed or unexposed applications. Within the latter category, cold-rolled sheet is specified either "temper rolled" or "annealed last."	Sheet	SS				
		25	25	42	26	1.68
		30	30	45	24	1.50
		33 Type 1	33	48	22	1.45
		33 Type 2	33	48	22	1.45
		40 Type 1	40	52	20	1.30
		40 Type 2	40	52	20	1.30
		80	80	82	--	--
		HSLAS Class 1				
		45	45	60	22	1.33
		50	50	65	20	1.30
		55	55	70	18	1.27
		60	60	75	16	1.25
		65	65	80	15	1.23
		70	70	85	14	1.21
		HSLAS Class 2				
		45	45	55	22	1.22
		50	50	60	20	1.20
		55	55	65	18	1.18
		60	60	70	16	1.17
		65	65	75	15	1.15
		70	70	80	14	1.14
		HSLAS-F				
		50	50	60	22	1.20
		60	60	70	18	1.17
		70	70	80	16	1.14
		80	80	90	14	1.12

Table 1.2

Summary Of Scope And Principle Tensile Properties
ASTM Specifications for Referenced Steels

ASTM Designation SCOPE (After ASTM)	PRODUCT	GRADE	F_y ksi (min)	F_u ksi (min/max)	Percent elongation in 2 inches (min)	$\frac{F_{u(min)}}{F_{y(min)}}$
A1011/A1011M-05a This specification covers hot-rolled structural, high-strength low-alloy, and high-strength low-alloy with improved formability steel sheet and strip, in coils and cut lengths. The product is produced in a number of designations including structural steel (SS), high-strength low-alloy steel (HSLAS), and high-strength low-alloy steel with improved formability (HSLAS-F). HSLAS-F steel has improved formability compared to HSLAS. The steel is fully deoxidized, made to fine grain practice, and includes microalloying elements such as columbium, vanadium, and zirconium. The steel is treated to achieve inclusion control. * Specified value varies with thickness range.	Sheet	SS				
		30	30	49	25-21*	1.63
		33	33	52	23-18*	1.62
		36 Type 1	36	53	22-17*	1.47
		36 Type 2	36	58/80	21-16*	1.61
		40	40	55	21-15*	1.38
		45	45	60	19-13*	1.33
		50	50	65	17-11*	1.30
		55	55	70	15-9*	1.27
		HSLAS Class 1				
		45	45	60	25-23*	1.33
		50	50	65	22-20*	1.30
		55	55	70	20-18*	1.27
		60	60	75	18-16*	1.25
		65	65	80	16-14*	1.23
		70	70	85	14-12*	1.21
		HSLAS Class 2				
		45	45	55	25-23*	1.22
		50	50	60	22-20*	1.20
		55	55	65	20-18*	1.18
		60	60	70	18-16*	1.17
		65	65	75	16-14*	1.15
		70	70	80	14-12*	1.14
		HSLAS-F				
		50	50	60	24-22*	1.20
		60	60	70	22-20*	1.17
		70	70	80	20-18*	1.14
		80	80	90	18-16*	1.12
A1039/A1039M-04 This specification covers commercial and structural steel sheet in coils and cut lengths produced by the twin-roll casting process. Designations are as follows: Commercial steel (CS Types A and B) and structural steel (SS grades 40 to 80). Mechanical properties are specified for SS grades but are non-mandatory for CS grades. * Limited per Section A2.3.2 if 10% minimum elongation requirement is not met	Sheet	SS				
		40	40	55	20-15*	1.38
		50	50	65	16-11*	1.30
		55*	55	70	14-9*	1.27
		60*	60	70	13-8*	1.17
		70*	70	80	12-7*	1.14
		80*	80	90	11-6*	1.12

1.3 Material Thickness

Historically, sheet and strip steels have been ordered from the steel producer using one of the following systems to specify thickness:

Minimum Thickness: When ordered to a minimum thickness, all thickness tolerances are over (+) and nothing under (-). Steel is generally ordered to a minimum thickness when the design is based on minimum strength requirements that depend on having a guaranteed minimum thickness for the sheet product.

Nominal Thickness: When ordered to a nominal thickness, thickness tolerances are equally divided between over (+) and under (-). Steel is generally ordered to a nominal thickness when the equipment to be used to process the material is designed for a certain thickness.

Gauge (Gage) Thickness: Gauge thickness is an obsolete method of specifying sheet and strip steel thickness. Gauge numbers are only a very rough approximation of steel thickness and should not be used to order, design or specify any sheet or strip steel product.

Hot-dip coated sheet products are typically specified by total product thickness, including the coating. The relevant ASTM specifications for the various coated sheet products include values for the thickness of the coating itself.

Design Thickness

The steel thickness used in design should be the thickness of the uncoated base steel sheet or strip. Coatings such as paint or zinc add little or no structural strength and should not be included in the design thickness.

Delivered Minimum Thickness

Since there are tolerances in either of the two acceptable methods of ordering sheet and strip steel thickness, it would be unreasonable to expect the delivered minimum thickness of a cold-formed steel product to exactly match the design thickness. *Specification* provisions cover minor negative thickness tolerances. Thus, 95 percent of the design thickness has been set as the minimum delivered thickness of a cold-formed steel product.

If the delivered minimum thickness is less than 95 percent of the design thickness, an analysis should be performed to determine if the delivered product is adequate to meet its intended purpose. Generally, thickness measurements may be made anywhere across the width of the sheet, but not closer to the edges than the minimum distances specified in the relevant ASTM specifications. Thickness at bends, such as corners, may be less than 95 percent of design thickness, due to cold-forming effects, and still be acceptable.

SECTION 2 - REPRESENTATIVE COLD-FORMED STEEL SECTIONS

2.1 Representative Versus Actual Sections

The cross-sections defined in Tables I-1 to I-8 are intended to be representative of some of the sections in use by, or available from, manufacturers and fabricators. The specific sections listed in these tables are not necessarily stock sections. They are included primarily as a guide in the design of cold-formed steel structural members. Although these sections are useful for preliminary design, designers should consult the literature of cold-formed product providers for actual section property information when specifying cold-formed products.

Two different naming conventions are used throughout the *Manual*. Standard stud and track sections are identified using the SSMA naming convention. Sections other than studs and tracks are identified by a convention developed by AISI for use in this *Manual*.

2.1.1 SSMA Stud and Track Section Nomenclature

Standard studs and tracks produced by members of the Steel Stud Manufacturers Association (SSMA) are identified in this *Manual* by the SSMA identification code. The codes are formed by concatenating the following information:

1. Depth in 1/100th inches. For studs, the depth is the outside depth. For tracks, the depth is the inside depth (the depth of the stud the track fits over).
2. Style: S =Stud (C-Section with Lips), T = Track (C-Section without Lips)
3. Flange Width in 1/100th inches
4. "-"
5. Minimum base material thickness (95% of design thickness) in 1/1000th inches

For example, a section with the designation 600S162-54 is a stud (C-section with lips), with a depth of 6 inches, a flange width of 1 5/8 inches and a minimum thickness of 0.054 inches. Other details, such as bend radii and lip lengths are found in Tables I-2 and I-3.

This naming convention is an industry standard.

2.1.2 Other Section Nomenclature

The naming convention used for the other representative sections was developed only to simplify the charts, tables and example problems throughout the *Manual*. The section names are formed by concatenating the following information:

1. Depth in inches
2. Section Profile: C = C-Section, Z = Z-Section, L = Equal Leg Angle, H = Hat Section
3. Code for Stiffened or Unstiffened Flanges: S = Stiffened, U = Unstiffened
4. Flange Width in inches
5. "x"
6. Thickness in 1/1000th inches

For example, a section with the designation 9CS3x075 is a C-Section with stiffened lips, with a depth of 9 inches, a flange width of 3 inches and a thickness of 0.075 inches. Other details, such as bend radii and lip lengths are found in Tables I-1 and I-4 through I-8.

This naming convention is not an industry standard. Individual manufacturers and industry groups have adopted their own systems, and these systems should be used when specifying actual products.

2.2 Notes On Tables

(a) Tabulated section properties are shown to three significant figures, while dimensions are given to three decimal places. However, in some cases space limitations made it impractical to adhere strictly to this guideline.

(b) The weight of these sections is calculated based on a steel weight of 40.8 pounds per square foot per inch thickness.

(c) Where they apply, the algebraic formulae presented in Section 3 of Part I formed the basis of the calculations for these tables.

(d) Tables I-1 to I-8 inclusive are Gross Section Property Tables. Effective section properties can be found in Parts II and III for beams and columns, respectively.

(e) In Table I-8, the orientation of the x-axis is vertical to be consistent with the provisions of *Specification* Section C3.1.2.1 which defines the x-axis as the axis of symmetry for singly-symmetric section.

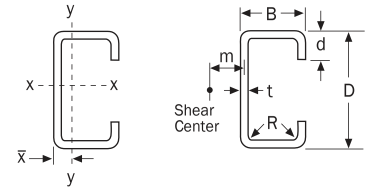
(f) Section dimensions are defined in the figures provided in each table. Section properties are defined in the *Specification*, Symbols and Definitions.

2.3 Gross Section Property Tables

Table I - 1

Gross Section Properties

C-Sections With Lips

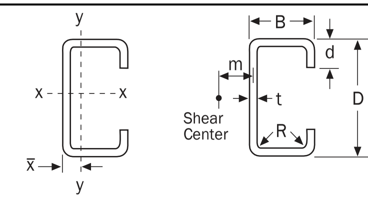


ID	Dimensions							Properties of Full Section														
								Axis x-x			Axis y-y											
	D	B	t	d	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
12CS4x105	12.000	4.000	0.105	0.885	0.1875	2.20	7.48	47.5	7.91	4.65	4.27	1.45	1.39	1.05	1.64	0.00808	122	6.74	5.52	-2.63
12CS4x085	12.000	4.000	0.085	0.836	0.1875	1.78	6.05	38.6	6.43	4.66	3.45	1.17	1.39	1.03	1.63	0.00429	98.5	6.76	5.52	-2.62
12CS4x070	12.000	4.000	0.070	0.800	0.1875	1.47	4.98	31.9	5.31	4.66	2.84	0.955	1.39	1.02	1.63	0.00239	80.8	6.77	5.52	-2.62
12CS3.5x105	12.000	3.500	0.105	0.885	0.1875	2.09	7.12	43.8	7.29	4.57	3.07	1.17	1.21	0.879	1.41	0.00769	89.1	6.81	5.23	-2.23
12CS3.5x085	12.000	3.500	0.085	0.836	0.1875	1.69	5.76	35.6	5.93	4.58	2.48	0.942	1.21	0.865	1.40	0.00408	71.7	6.83	5.23	-2.22
12CS3.5x070	12.000	3.500	0.070	0.800	0.1875	1.40	4.75	29.4	4.90	4.59	2.04	0.773	1.21	0.855	1.40	0.00228	58.8	6.85	5.24	-2.22
12CS2.5x105	12.000	2.500	0.105	0.885	0.1875	1.88	6.40	36.3	6.05	4.39	1.34	0.692	0.843	0.567	0.951	0.00692	40.4	7.56	4.71	-1.47
12CS2.5x085	12.000	2.500	0.085	0.836	0.1875	1.52	5.18	29.5	4.92	4.40	1.08	0.557	0.843	0.555	0.947	0.00367	32.6	7.59	4.71	-1.46
12CS2.5x070	12.000	2.500	0.070	0.800	0.1875	1.26	4.27	24.4	4.06	4.41	0.893	0.457	0.844	0.546	0.943	0.00205	26.8	7.61	4.72	-1.45
10CS4x105	10.000	4.000	0.105	0.885	0.1875	1.99	6.76	31.0	6.20	3.95	4.04	1.42	1.43	1.15	1.73	0.00731	81.7	5.70	5.06	-2.83
10CS4x085	10.000	4.000	0.085	0.836	0.1875	1.61	5.47	25.2	5.05	3.96	3.27	1.14	1.43	1.14	1.72	0.00388	65.7	5.72	5.07	-2.82
10CS4x070	10.000	4.000	0.070	0.800	0.1875	1.33	4.51	20.9	4.17	3.97	2.69	0.937	1.43	1.13	1.72	0.00217	53.8	5.73	5.07	-2.81
10CS4x065	10.000	4.000	0.065	0.788	0.1875	1.23	4.19	19.4	3.88	3.97	2.50	0.869	1.43	1.13	1.71	0.00173	49.9	5.73	5.07	-2.81
10CS3.5x105	10.000	3.500	0.105	0.885	0.1875	1.88	6.40	28.5	5.69	3.89	2.91	1.15	1.24	0.971	1.49	0.00692	59.5	5.59	4.74	-2.41
10CS3.5x085	10.000	3.500	0.085	0.836	0.1875	1.52	5.18	23.1	4.63	3.90	2.35	0.926	1.24	0.957	1.48	0.00367	47.8	5.61	4.74	-2.40
10CS3.5x070	10.000	3.500	0.070	0.800	0.1875	1.26	4.27	19.1	3.83	3.90	1.94	0.759	1.24	0.947	1.48	0.00205	39.2	5.63	4.74	-2.39
10CS3.5x065	10.000	3.500	0.065	0.788	0.1875	1.17	3.96	17.8	3.56	3.91	1.80	0.704	1.24	0.943	1.48	0.00164	36.3	5.63	4.74	-2.39
10CS2.5x105	10.000	2.500	0.105	0.885	0.1875	1.67	5.69	23.3	4.66	3.73	1.28	0.683	0.873	0.632	1.02	0.00615	27.0	5.78	4.15	-1.60
10CS2.5x085	10.000	2.500	0.085	0.836	0.1875	1.35	4.61	19.0	3.79	3.74	1.03	0.550	0.874	0.619	1.02	0.00326	21.8	5.81	4.16	-1.59
10CS2.5x070	10.000	2.500	0.070	0.800	0.1875	1.12	3.79	15.7	3.14	3.75	0.852	0.451	0.874	0.610	1.01	0.00182	17.9	5.83	4.16	-1.59
10CS2.5x065	10.000	2.500	0.065	0.788	0.1875	1.04	3.52	14.6	2.92	3.75	0.792	0.418	0.874	0.607	1.01	0.00146	16.6	5.84	4.17	-1.59
10CS2x105	10.000	2.000	0.105	0.885	0.1875	1.57	5.33	20.7	4.15	3.64	0.739	0.486	0.687	0.478	0.791	0.00576	16.1	6.28	3.90	-1.22
10CS2x085	10.000	2.000	0.085	0.836	0.1875	1.27	4.32	16.9	3.38	3.65	0.601	0.392	0.688	0.466	0.788	0.00306	13.0	6.32	3.90	-1.21
10CS2x070	10.000	2.000	0.070	0.800	0.1875	1.05	3.56	14.0	2.79	3.65	0.496	0.322	0.689	0.457	0.785	0.00171	10.7	6.34	3.91	-1.21
10CS2x065	10.000	2.000	0.065	0.788	0.1875	0.971	3.30	13.0	2.60	3.66	0.461	0.298	0.689	0.454	0.784	0.00137	9.91	6.35	3.91	-1.21

Table I - 1

Gross Section Properties

C-Sections With Lips

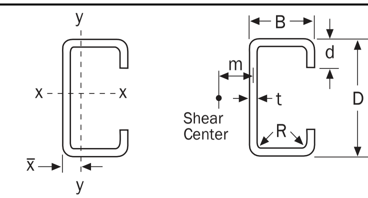


ID	Dimensions							Properties of Full Section												
								Axis x-x			Axis y-y									
	D	B	t	d	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
9CS2.5x105	9.000	2.500	0.105	0.885	0.1875	1.57	5.33	18.1	4.02	3.40	1.24	0.676	0.888	0.670	1.06	0.00576	21.5	5.03	3.89	-1.68
9CS2.5x085	9.000	2.500	0.085	0.836	0.1875	1.27	4.32	14.7	3.27	3.41	1.00	0.545	0.889	0.658	1.06	0.00306	17.3	5.06	3.90	-1.67
9CS2.5x070	9.000	2.500	0.070	0.800	0.1875	1.05	3.56	12.2	2.71	3.41	0.828	0.447	0.890	0.648	1.05	0.00171	14.2	5.09	3.90	-1.66
9CS2.5x065	9.000	2.500	0.065	0.788	0.1875	0.971	3.30	11.3	2.52	3.42	0.769	0.415	0.890	0.645	1.05	0.00137	13.1	5.09	3.90	-1.66
9CS2.5x059	9.000	2.500	0.059	0.773	0.1875	0.881	3.00	10.3	2.29	3.42	0.698	0.376	0.890	0.641	1.05	0.00102	11.9	5.10	3.90	-1.66
8CS4x105	8.000	4.000	0.105	0.885	0.1875	1.78	6.05	18.6	4.64	3.23	3.76	1.38	1.45	1.28	1.83	0.00654	50.4	4.92	4.68	-3.06
8CS4x085	8.000	4.000	0.085	0.836	0.1875	1.44	4.90	15.1	3.78	3.24	3.04	1.11	1.45	1.27	1.83	0.00347	40.4	4.93	4.68	-3.05
8CS4x070	8.000	4.000	0.070	0.800	0.1875	1.19	4.03	12.5	3.13	3.25	2.50	0.913	1.45	1.26	1.82	0.00194	33.1	4.94	4.68	-3.04
8CS4x065	8.000	4.000	0.065	0.788	0.1875	1.10	3.74	11.6	2.91	3.25	2.33	0.847	1.45	1.25	1.82	0.00155	30.6	4.94	4.68	-3.04
8CS4x059	8.000	4.000	0.059	0.773	0.1875	0.999	3.40	10.6	2.65	3.25	2.11	0.767	1.45	1.25	1.81	0.00116	27.7	4.95	4.68	-3.03
8CS3.5x105	8.000	3.500	0.105	0.885	0.1875	1.67	5.69	16.9	4.23	3.18	2.71	1.12	1.27	1.09	1.59	0.00615	36.7	4.66	4.31	-2.62
8CS3.5x085	8.000	3.500	0.085	0.836	0.1875	1.35	4.61	13.8	3.45	3.19	2.19	0.904	1.27	1.07	1.58	0.00326	29.5	4.68	4.31	-2.61
8CS3.5x070	8.000	3.500	0.070	0.800	0.1875	1.12	3.79	11.4	2.85	3.20	1.81	0.741	1.27	1.06	1.58	0.00182	24.1	4.69	4.32	-2.60
8CS3.5x065	8.000	3.500	0.065	0.788	0.1875	1.04	3.52	10.6	2.65	3.20	1.68	0.687	1.27	1.06	1.57	0.00146	22.3	4.69	4.32	-2.60
8CS3.5x059	8.000	3.500	0.059	0.773	0.1875	0.940	3.20	9.65	2.41	3.20	1.52	0.623	1.27	1.05	1.57	0.00109	20.2	4.70	4.32	-2.60
8CS2.5x105	8.000	2.500	0.105	0.885	0.1875	1.46	4.98	13.6	3.41	3.05	1.19	0.669	0.903	0.715	1.10	0.00538	16.7	4.39	3.64	-1.77
8CS2.5x085	8.000	2.500	0.085	0.836	0.1875	1.18	4.03	11.1	2.78	3.06	0.969	0.539	0.904	0.702	1.10	0.00285	13.4	4.42	3.65	-1.76
8CS2.5x070	8.000	2.500	0.070	0.800	0.1875	0.976	3.32	9.21	2.30	3.07	0.800	0.442	0.905	0.692	1.09	0.00159	11.0	4.44	3.65	-1.75
8CS2.5x065	8.000	2.500	0.065	0.788	0.1875	0.906	3.08	8.57	2.14	3.08	0.743	0.410	0.905	0.689	1.09	0.00128	10.2	4.44	3.65	-1.75
8CS2.5x059	8.000	2.500	0.059	0.773	0.1875	0.822	2.80	7.79	1.95	3.08	0.674	0.372	0.906	0.685	1.09	0.000954	9.22	4.45	3.65	-1.75
8CS2x105	8.000	2.000	0.105	0.885	0.1875	1.36	4.62	12.0	3.00	2.97	0.696	0.478	0.716	0.544	0.863	0.00499	9.95	4.50	3.35	-1.36
8CS2x085	8.000	2.000	0.085	0.836	0.1875	1.10	3.74	9.79	2.45	2.98	0.566	0.385	0.717	0.532	0.859	0.00265	8.01	4.53	3.35	-1.35
8CS2x070	8.000	2.000	0.070	0.800	0.1875	0.906	3.08	8.11	2.03	2.99	0.467	0.316	0.718	0.523	0.856	0.00148	6.58	4.56	3.36	-1.34
8CS2x065	8.000	2.000	0.065	0.788	0.1875	0.841	2.86	7.54	1.89	3.00	0.435	0.294	0.719	0.520	0.855	0.00118	6.10	4.57	3.36	-1.34
8CS2x059	8.000	2.000	0.059	0.773	0.1875	0.763	2.60	6.86	1.72	3.00	0.395	0.266	0.719	0.516	0.854	0.000886	5.53	4.58	3.36	-1.34
7CS4x105	7.000	4.000	0.105	0.885	0.1875	1.67	5.69	13.7	3.91	2.86	3.59	1.36	1.46	1.36	1.89	0.00615	38.1	4.63	4.53	-3.20
7CS4x085	7.000	4.000	0.085	0.836	0.1875	1.35	4.61	11.2	3.19	2.87	2.91	1.09	1.47	1.35	1.88	0.00326	30.5	4.64	4.53	-3.19
7CS4x070	7.000	4.000	0.070	0.800	0.1875	1.12	3.79	9.24	2.64	2.88	2.39	0.898	1.47	1.33	1.88	0.00182	24.9	4.64	4.53	-3.18
7CS4x065	7.000	4.000	0.065	0.788	0.1875	1.04	3.52	8.60	2.46	2.88	2.22	0.833	1.47	1.33	1.87	0.00146	23.1	4.65	4.53	-3.17
7CS4x059	7.000	4.000	0.059	0.773	0.1875	0.940	3.20	7.82	2.23	2.88	2.02	0.754	1.46	1.33	1.87	0.00109	20.9	4.65	4.53	-3.17

Table I - 1

Gross Section Properties

C-Sections With Lips



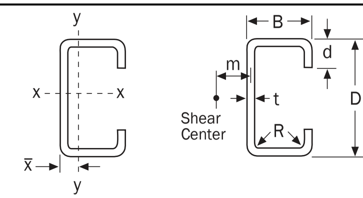
ID	Dimensions							Properties of Full Section												
								Axis x-x			Axis y-y									
	D	B	t	d	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
7CS2.5x105	7.000	2.500	0.105	0.885	0.1875	1.36	4.62	9.94	2.84	2.71	1.15	0.660	0.918	0.766	1.15	0.00499	12.6	3.85	3.41	-1.86
7CS2.5x085	7.000	2.500	0.085	0.836	0.1875	1.10	3.74	8.11	2.32	2.72	0.929	0.532	0.919	0.753	1.15	0.00265	10.1	3.87	3.42	-1.86
7CS2.5x070	7.000	2.500	0.070	0.800	0.1875	0.906	3.08	6.72	1.92	2.72	0.767	0.437	0.920	0.743	1.14	0.00148	8.27	3.89	3.42	-1.85
7CS2.5x065	7.000	2.500	0.065	0.788	0.1875	0.841	2.86	6.25	1.79	2.73	0.713	0.405	0.921	0.740	1.14	0.00118	7.66	3.90	3.42	-1.85
7CS2.5x059	7.000	2.500	0.059	0.773	0.1875	0.763	2.60	5.69	1.63	2.73	0.647	0.367	0.921	0.735	1.14	0.000886	6.93	3.90	3.42	-1.84
6CS4x105	6.000	4.000	0.105	0.885	0.1875	1.57	5.33	9.64	3.21	2.48	3.40	1.33	1.47	1.45	1.95	0.00576	27.8	4.40	4.42	-3.35
6CS4x085	6.000	4.000	0.085	0.836	0.1875	1.27	4.32	7.88	2.63	2.49	2.75	1.07	1.47	1.43	1.94	0.00306	22.2	4.41	4.42	-3.34
6CS4x070	6.000	4.000	0.070	0.800	0.1875	1.05	3.56	6.53	2.18	2.50	2.27	0.879	1.47	1.42	1.94	0.00171	18.1	4.41	4.41	-3.32
6CS4x065	6.000	4.000	0.065	0.788	0.1875	0.971	3.30	6.08	2.03	2.50	2.11	0.815	1.47	1.42	1.94	0.00137	16.7	4.41	4.41	-3.32
6CS4x059	6.000	4.000	0.059	0.773	0.1875	0.881	3.00	5.53	1.84	2.51	1.91	0.739	1.47	1.41	1.93	0.00102	15.1	4.42	4.41	-3.32
6CS2.5x105	6.000	2.500	0.105	0.885	0.1875	1.25	4.26	6.91	2.30	2.35	1.09	0.649	0.931	0.826	1.20	0.00461	9.20	3.40	3.21	-1.98
6CS2.5x085	6.000	2.500	0.085	0.836	0.1875	1.01	3.45	5.65	1.88	2.36	0.883	0.523	0.933	0.812	1.20	0.00244	7.36	3.43	3.21	-1.97
6CS2.5x070	6.000	2.500	0.070	0.800	0.1875	0.836	2.84	4.69	1.56	2.37	0.729	0.429	0.934	0.802	1.19	0.00136	6.01	3.44	3.21	-1.96
6CS2.5x065	6.000	2.500	0.065	0.788	0.1875	0.776	2.64	4.36	1.45	2.37	0.677	0.398	0.934	0.799	1.19	0.00109	5.56	3.45	3.21	-1.96
6CS2.5x059	6.000	2.500	0.059	0.773	0.1875	0.704	2.40	3.97	1.32	2.37	0.615	0.361	0.935	0.795	1.19	0.000817	5.03	3.45	3.21	-1.95
4CS4x105	4.000	4.000	0.105	0.885	0.1875	1.36	4.62	3.87	1.93	1.69	2.92	1.25	1.47	1.66	2.10	0.00499	12.9	4.15	4.33	-3.71
4CS4x085	4.000	4.000	0.085	0.836	0.1875	1.10	3.74	3.18	1.59	1.70	2.37	1.01	1.47	1.65	2.09	0.00265	10.2	4.16	4.32	-3.69
4CS4x070	4.000	4.000	0.070	0.800	0.1875	0.906	3.08	2.64	1.32	1.71	1.96	0.828	1.47	1.64	2.08	0.00148	8.25	4.16	4.31	-3.68
4CS4x065	4.000	4.000	0.065	0.788	0.1875	0.841	2.86	2.46	1.23	1.71	1.82	0.767	1.47	1.63	2.08	0.00118	7.62	4.16	4.31	-3.67
4CS4x059	4.000	4.000	0.059	0.773	0.1875	0.763	2.60	2.25	1.12	1.72	1.65	0.695	1.47	1.63	2.07	0.000886	6.87	4.16	4.31	-3.67
4CS2.5x105	4.000	2.500	0.105	0.885	0.1875	1.04	3.55	2.67	1.34	1.60	0.936	0.617	0.947	0.981	1.33	0.00383	4.30	2.83	2.92	-2.26
4CS2.5x085	4.000	2.500	0.085	0.836	0.1875	0.845	2.87	2.20	1.10	1.61	0.762	0.497	0.950	0.967	1.32	0.00203	3.40	2.84	2.92	-2.24
4CS2.5x070	4.000	2.500	0.070	0.800	0.1875	0.696	2.37	1.83	0.917	1.62	0.630	0.408	0.952	0.957	1.31	0.00114	2.75	2.85	2.92	-2.24
4CS2.5x065	4.000	2.500	0.065	0.788	0.1875	0.646	2.20	1.71	0.855	1.63	0.586	0.379	0.952	0.953	1.31	0.000910	2.54	2.86	2.92	-2.23
4CS2.5x059	4.000	2.500	0.059	0.773	0.1875	0.586	1.99	1.56	0.780	1.63	0.532	0.343	0.953	0.949	1.31	0.000680	2.29	2.86	2.92	-2.23
4CS2x105	4.000	2.000	0.105	0.979	0.1875	0.958	3.26	2.30	1.15	1.55	0.576	0.475	0.775	0.788	1.10	0.00352	2.91	2.42	2.53	-1.84
4CS2x085	4.000	2.000	0.085	0.930	0.1875	0.776	2.64	1.89	0.947	1.56	0.471	0.384	0.779	0.775	1.10	0.00187	2.31	2.44	2.53	-1.83
4CS2x070	4.000	2.000	0.070	0.894	0.1875	0.639	2.17	1.58	0.791	1.57	0.390	0.316	0.782	0.766	1.09	0.00104	1.87	2.46	2.53	-1.82
4CS2x065	4.000	2.000	0.065	0.881	0.1875	0.593	2.02	1.47	0.737	1.58	0.363	0.294	0.782	0.763	1.09	0.000835	1.73	2.46	2.53	-1.82
4CS2x059	4.000	2.000	0.059	0.867	0.1875	0.538	1.83	1.35	0.673	1.58	0.331	0.266	0.784	0.759	1.09	0.000625	1.56	2.47	2.54	-1.82

Table I - 2

Gross Section Properties

SSMA Studs

C-Sections With Lips



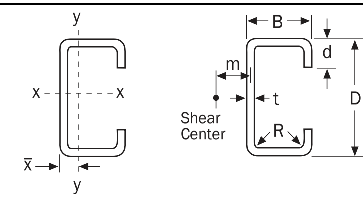
ID	Dimensions							Properties of Full Section													
								Axis x-x			Axis y-y										
	D	B	t	d	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
1200S250-97	12.000	2.500	0.1017	0.625	0.1526	1.78	6.05	34.0	5.67	4.37	1.12	0.565	0.794	0.513	0.867	0.00613	32.7	7.93	4.64	-1.33
1200S250-68	12.000	2.500	0.0713	0.625	0.1070	1.26	4.30	24.5	4.08	4.40	0.836	0.421	0.813	0.513	0.884	0.00214	24.0	7.69	4.68	-1.36
1200S250-54*	12.000	2.500	0.0566	0.625	0.0849	1.01	3.43	19.7	3.28	4.42	0.683	0.344	0.823	0.513	0.892	0.00108	19.5	7.58	4.70	-1.38
1200S200-97	12.000	2.000	0.1017	0.625	0.1526	1.68	5.70	30.4	5.07	4.26	0.635	0.392	0.615	0.381	0.656	0.00578	19.1	9.11	4.41	-0.987
1200S200-68	12.000	2.000	0.0713	0.625	0.1070	1.19	4.05	21.9	3.66	4.29	0.479	0.296	0.634	0.380	0.673	0.00202	14.2	8.74	4.46	-1.02
1200S200-54*	12.000	2.000	0.0566	0.625	0.0849	0.953	3.24	17.7	2.94	4.31	0.394	0.243	0.643	0.379	0.681	0.00102	11.6	8.58	4.47	-1.03
1200S162-97	12.000	1.625	0.1017	0.500	0.1526	1.58	5.36	27.0	4.49	4.14	0.332	0.245	0.459	0.272	0.470	0.00543	10.3	11.3	4.22	-0.691
1200S162-68	12.000	1.625	0.0713	0.500	0.1070	1.12	3.81	19.5	3.25	4.17	0.255	0.188	0.477	0.269	0.485	0.00190	7.74	10.6	4.26	-0.719
1200S162-54*	12.000	1.625	0.0566	0.500	0.0849	0.896	3.05	15.7	2.62	4.19	0.212	0.156	0.486	0.268	0.493	0.000957	6.34	10.3	4.28	-0.732
1000S250-97	10.000	2.500	0.1017	0.625	0.1526	1.58	5.36	21.8	4.37	3.72	1.07	0.557	0.825	0.573	0.932	0.00543	21.6	6.03	4.08	-1.45
1000S250-68	10.000	2.500	0.0713	0.625	0.1070	1.12	3.81	15.8	3.15	3.75	0.799	0.415	0.844	0.574	0.950	0.00190	15.9	5.88	4.12	-1.49
1000S250-54	10.000	2.500	0.0566	0.625	0.0849	0.896	3.05	12.7	2.54	3.76	0.653	0.339	0.854	0.575	0.958	0.000957	12.9	5.81	4.14	-1.51
1000S250-43*	10.000	2.500	0.0451	0.625	0.0712	0.718	2.44	10.2	2.04	3.77	0.531	0.276	0.861	0.575	0.965	0.000486	10.5	5.76	4.16	-1.52
1000S200-97	10.000	2.000	0.1017	0.625	0.1526	1.47	5.01	19.3	3.87	3.62	0.610	0.388	0.643	0.427	0.711	0.00508	12.7	6.68	3.84	-1.09
1000S200-68	10.000	2.000	0.0713	0.625	0.1070	1.05	3.57	14.0	2.80	3.65	0.460	0.292	0.662	0.427	0.729	0.00178	9.40	6.44	3.88	-1.12
1000S200-54	10.000	2.000	0.0566	0.625	0.0849	0.839	2.85	11.3	2.26	3.67	0.378	0.240	0.671	0.427	0.737	0.000896	7.67	6.33	3.90	-1.14
1000S200-43*	10.000	2.000	0.0451	0.625	0.0712	0.672	2.29	9.09	1.82	3.68	0.309	0.196	0.677	0.426	0.743	0.000456	6.24	6.26	3.91	-1.15
1000S162-97	10.000	1.625	0.1017	0.500	0.1526	1.37	4.67	17.0	3.39	3.52	0.320	0.243	0.483	0.305	0.514	0.00473	6.83	8.05	3.63	-0.768
1000S162-68	10.000	1.625	0.0713	0.500	0.1070	0.978	3.33	12.3	2.47	3.55	0.247	0.187	0.502	0.303	0.531	0.00166	5.12	7.62	3.67	-0.798
1000S162-54	10.000	1.625	0.0566	0.500	0.0849	0.783	2.66	9.95	1.99	3.57	0.204	0.155	0.511	0.302	0.538	0.000836	4.20	7.43	3.69	-0.812
1000S162-43*	10.000	1.625	0.0451	0.500	0.0712	0.627	2.13	8.03	1.61	3.58	0.168	0.127	0.518	0.301	0.545	0.000425	3.43	7.31	3.71	-0.823
800S250-97	8.000	2.500	0.1017	0.625	0.1526	1.37	4.67	12.8	3.20	3.05	1.01	0.546	0.858	0.650	1.01	0.00473	13.1	4.54	3.56	-1.61
800S250-68	8.000	2.500	0.0713	0.625	0.1070	0.978	3.33	9.26	2.32	3.08	0.752	0.407	0.877	0.653	1.03	0.00166	9.65	4.45	3.60	-1.64
800S250-54	8.000	2.500	0.0566	0.625	0.0849	0.783	2.66	7.47	1.87	3.09	0.614	0.333	0.886	0.654	1.04	0.000836	7.85	4.42	3.62	-1.66
800S250-43	8.000	2.500	0.0451	0.625	0.0712	0.627	2.13	6.02	1.50	3.10	0.500	0.271	0.893	0.654	1.04	0.000425	6.37	4.39	3.63	-1.68

Table I - 2

Gross Section Properties

SSMA Studs

C-Sections With Lips



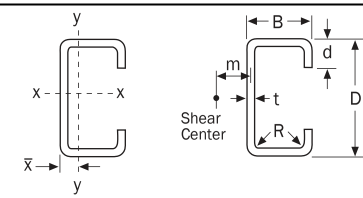
ID	Dimensions							Properties of Full Section												
								Axis x-x			Axis y-y									
	D	B	t	d	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
800S200-97	8.000	2.000	0.1017	0.625	0.1526	1.27	4.32	11.2	2.80	2.97	0.577	0.381	0.674	0.487	0.777	0.00438	7.68	4.76	3.28	-1.21
800S200-68	8.000	2.000	0.0713	0.625	0.1070	0.907	3.08	8.14	2.04	3.00	0.435	0.288	0.692	0.488	0.796	0.00154	5.71	4.62	3.32	-1.25
800S200-54	8.000	2.000	0.0566	0.625	0.0849	0.726	2.47	6.57	1.64	3.01	0.357	0.236	0.701	0.489	0.804	0.000775	4.66	4.56	3.34	-1.27
800S200-43	8.000	2.000	0.0451	0.625	0.0712	0.582	1.98	5.30	1.33	3.02	0.292	0.193	0.708	0.489	0.811	0.000395	3.80	4.52	3.35	-1.28
800S200-33*	8.000	2.000	0.0346	0.625	0.0765	0.448	1.52	4.10	1.02	3.02	0.227	0.150	0.712	0.488	0.817	0.000179	2.97	4.52	3.36	-1.29
800S162-97	8.000	1.625	0.1017	0.500	0.1526	1.17	3.97	9.71	2.43	2.88	0.305	0.239	0.511	0.349	0.568	0.00403	4.11	5.48	3.05	-0.866
800S162-68	8.000	1.625	0.0713	0.500	0.1070	0.836	2.84	7.09	1.77	2.91	0.235	0.184	0.530	0.349	0.586	0.00142	3.09	5.22	3.09	-0.899
800S162-54	8.000	1.625	0.0566	0.500	0.0849	0.670	2.28	5.74	1.43	2.93	0.195	0.152	0.539	0.348	0.594	0.000715	2.54	5.11	3.11	-0.914
800S162-43	8.000	1.625	0.0451	0.500	0.0712	0.537	1.83	4.63	1.16	2.94	0.160	0.125	0.546	0.348	0.601	0.000364	2.08	5.04	3.13	-0.926
800S162-33*	8.000	1.625	0.0346	0.500	0.0765	0.414	1.41	3.58	0.896	2.94	0.125	0.0980	0.550	0.347	0.607	0.000165	1.63	5.03	3.14	-0.936
800S137-97	8.000	1.375	0.1017	0.375	0.1526	1.09	3.72	8.60	2.15	2.81	0.170	0.152	0.394	0.258	0.423	0.00377	2.35	6.57	2.90	-0.630
800S137-68	8.000	1.375	0.0713	0.375	0.1070	0.782	2.66	6.30	1.58	2.84	0.134	0.120	0.414	0.257	0.440	0.00133	1.79	6.12	2.94	-0.661
800S137-54	8.000	1.375	0.0566	0.375	0.0849	0.627	2.13	5.11	1.28	2.86	0.112	0.100	0.423	0.256	0.448	0.000670	1.48	5.93	2.96	-0.676
800S137-43	8.000	1.375	0.0451	0.375	0.0712	0.503	1.71	4.13	1.03	2.87	0.0931	0.0831	0.430	0.255	0.454	0.000341	1.21	5.81	2.98	-0.687
800S137-33*	8.000	1.375	0.0346	0.375	0.0765	0.388	1.32	3.20	0.800	2.87	0.0732	0.0653	0.435	0.254	0.460	0.000155	0.957	5.78	2.99	-0.696
600S250-97	6.000	2.500	0.1017	0.625	0.1526	1.17	3.97	6.50	2.17	2.36	0.923	0.529	0.889	0.754	1.10	0.00403	6.95	3.46	3.10	-1.80
600S250-68	6.000	2.500	0.0713	0.625	0.1070	0.836	2.84	4.73	1.58	2.38	0.688	0.395	0.908	0.758	1.12	0.00142	5.15	3.43	3.14	-1.84
600S250-54	6.000	2.500	0.0566	0.625	0.0849	0.670	2.28	3.82	1.27	2.39	0.563	0.323	0.917	0.759	1.13	0.000715	4.19	3.41	3.16	-1.86
600S250-43	6.000	2.500	0.0451	0.625	0.0712	0.537	1.83	3.08	1.03	2.40	0.458	0.263	0.923	0.760	1.14	0.000364	3.41	3.41	3.18	-1.87
600S200-97	6.000	2.000	0.1017	0.625	0.1526	1.07	3.63	5.61	1.87	2.29	0.530	0.371	0.705	0.570	0.859	0.00368	4.08	3.35	2.77	-1.38
600S200-68	6.000	2.000	0.0713	0.625	0.1070	0.764	2.60	4.10	1.37	2.32	0.400	0.280	0.723	0.573	0.878	0.00130	3.05	3.29	2.81	-1.42
600S200-54	6.000	2.000	0.0566	0.625	0.0849	0.613	2.08	3.32	1.11	2.33	0.329	0.230	0.732	0.574	0.887	0.000655	2.49	3.26	2.83	-1.43
600S200-43	6.000	2.000	0.0451	0.625	0.0712	0.492	1.67	2.68	0.894	2.34	0.268	0.188	0.739	0.574	0.894	0.000334	2.03	3.25	2.84	-1.45
600S200-33	6.000	2.000	0.0346	0.625	0.0765	0.379	1.29	2.08	0.692	2.34	0.209	0.147	0.743	0.574	0.901	0.000151	1.59	3.25	2.86	-1.46
600S162-97	6.000	1.625	0.1017	0.500	0.1526	0.966	3.28	4.80	1.60	2.23	0.283	0.233	0.542	0.411	0.636	0.00333	2.15	3.56	2.50	-0.997
600S162-68	6.000	1.625	0.0713	0.500	0.1070	0.693	2.36	3.52	1.17	2.26	0.218	0.180	0.561	0.413	0.655	0.00117	1.63	3.43	2.54	-1.03
600S162-54	6.000	1.625	0.0566	0.500	0.0849	0.556	1.89	2.86	0.954	2.27	0.181	0.149	0.570	0.414	0.663	0.000594	1.34	3.38	2.56	-1.05
600S162-43	6.000	1.625	0.0451	0.500	0.0712	0.447	1.52	2.32	0.772	2.28	0.148	0.123	0.576	0.414	0.670	0.000303	1.10	3.35	2.58	-1.06
600S162-33	6.000	1.625	0.0346	0.500	0.0765	0.344	1.17	1.79	0.598	2.28	0.116	0.0959	0.581	0.413	0.677	0.000137	0.861	3.35	2.59	-1.07

Table I - 2

Gross Section Properties

SSMA Studs

C-Sections With Lips



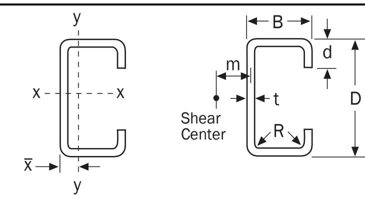
ID	Dimensions							Properties of Full Section													
								Axis x-x			Axis y-y										
	D	B	t	d	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
600S137-97	6.000	1.375	0.1017	0.375	0.1526	0.889	3.02	4.19	1.40	2.17	0.159	0.149	0.423	0.305	0.480	0.00307	1.22	4.05	2.33	-0.734
600S137-68	6.000	1.375	0.0713	0.375	0.1070	0.640	2.17	3.09	1.03	2.20	0.126	0.117	0.443	0.306	0.497	0.00108	0.930	3.82	2.37	-0.768
600S137-54	6.000	1.375	0.0566	0.375	0.0849	0.514	1.75	2.52	0.839	2.21	0.105	0.0984	0.452	0.306	0.506	0.000549	0.769	3.72	2.39	-0.784
600S137-43	6.000	1.375	0.0451	0.375	0.0712	0.413	1.40	2.04	0.681	2.22	0.0871	0.0815	0.459	0.306	0.513	0.000280	0.633	3.66	2.41	-0.796
600S137-33	6.000	1.375	0.0346	0.375	0.0765	0.318	1.08	1.58	0.527	2.23	0.0685	0.0641	0.464	0.305	0.519	0.000127	0.500	3.65	2.42	-0.807
550S162-68	5.500	1.625	0.0713	0.500	0.1070	0.657	2.24	2.86	1.04	2.09	0.213	0.178	0.568	0.433	0.675	0.00111	1.34	3.08	2.41	-1.07
550S162-54	5.500	1.625	0.0566	0.500	0.0849	0.528	1.80	2.32	0.845	2.10	0.176	0.148	0.577	0.434	0.684	0.000564	1.10	3.04	2.43	-1.09
550S162-43	5.500	1.625	0.0451	0.500	0.0712	0.424	1.44	1.88	0.685	2.11	0.145	0.122	0.584	0.435	0.691	0.000288	0.905	3.02	2.45	-1.10
550S162-33	5.500	1.625	0.0346	0.500	0.0765	0.327	1.11	1.46	0.530	2.11	0.113	0.0952	0.589	0.434	0.697	0.000130	0.713	3.02	2.46	-1.11
400S200-68	4.000	2.000	0.0713	0.625	0.1070	0.622	2.11	1.59	0.795	1.60	0.349	0.268	0.750	0.696	0.983	0.00105	1.32	2.45	2.41	-1.64
400S200-54	4.000	2.000	0.0566	0.625	0.0849	0.500	1.70	1.29	0.646	1.61	0.287	0.221	0.758	0.697	0.993	0.000534	1.08	2.45	2.43	-1.66
400S200-43	4.000	2.000	0.0451	0.625	0.0712	0.402	1.37	1.05	0.524	1.62	0.235	0.180	0.764	0.698	1.000	0.000272	0.886	2.45	2.45	-1.68
400S200-33	4.000	2.000	0.0346	0.625	0.0765	0.310	1.05	0.812	0.406	1.62	0.183	0.141	0.769	0.698	1.01	0.000124	0.697	2.46	2.46	-1.69
400S162-68	4.000	1.625	0.0713	0.500	0.1070	0.550	1.87	1.35	0.673	1.56	0.192	0.173	0.591	0.511	0.745	0.000933	0.677	2.27	2.07	-1.22
400S162-54	4.000	1.625	0.0566	0.500	0.0849	0.443	1.51	1.10	0.549	1.57	0.159	0.143	0.600	0.512	0.754	0.000473	0.560	2.25	2.09	-1.24
400S162-43	4.000	1.625	0.0451	0.500	0.0712	0.357	1.21	0.892	0.446	1.58	0.131	0.118	0.606	0.513	0.761	0.000242	0.460	2.25	2.11	-1.25
400S162-33	4.000	1.625	0.0346	0.500	0.0765	0.275	0.935	0.692	0.346	1.59	0.103	0.0923	0.611	0.512	0.768	0.000110	0.363	2.25	2.12	-1.26
400S137-68	4.000	1.375	0.0713	0.375	0.1070	0.497	1.69	1.17	0.583	1.53	0.112	0.113	0.475	0.384	0.574	0.000842	0.375	2.27	1.85	-0.922
400S137-54	4.000	1.375	0.0566	0.375	0.0849	0.401	1.36	0.953	0.477	1.54	0.0939	0.0949	0.484	0.385	0.583	0.000428	0.311	2.24	1.87	-0.940
400S137-43	4.000	1.375	0.0451	0.375	0.0712	0.323	1.10	0.776	0.388	1.55	0.0778	0.0787	0.491	0.386	0.591	0.000219	0.257	2.22	1.89	-0.954
400S137-33	4.000	1.375	0.0346	0.375	0.0765	0.249	0.847	0.603	0.302	1.56	0.0612	0.0618	0.496	0.385	0.597	0.0000994	0.204	2.22	1.90	-0.965
362S200-68	3.625	2.000	0.0713	0.625	0.1070	0.595	2.02	1.27	0.698	1.46	0.337	0.265	0.753	0.726	1.01	0.00101	1.09	2.35	2.36	-1.70
362S200-54	3.625	2.000	0.0566	0.625	0.0849	0.479	1.63	1.03	0.568	1.47	0.277	0.218	0.761	0.727	1.02	0.000511	0.896	2.35	2.38	-1.72
362S200-43	3.625	2.000	0.0451	0.625	0.0712	0.385	1.31	0.836	0.461	1.47	0.227	0.178	0.767	0.728	1.02	0.000261	0.734	2.35	2.40	-1.73
362S200-33	3.625	2.000	0.0346	0.625	0.0765	0.297	1.01	0.648	0.358	1.48	0.177	0.139	0.772	0.728	1.03	0.000118	0.577	2.36	2.41	-1.74
362S162-68	3.625	1.625	0.0713	0.500	0.1070	0.524	1.78	1.07	0.590	1.43	0.186	0.171	0.596	0.535	0.765	0.000887	0.552	2.12	2.00	-1.26
362S162-54	3.625	1.625	0.0566	0.500	0.0849	0.422	1.43	0.873	0.482	1.44	0.154	0.142	0.605	0.536	0.774	0.000451	0.457	2.11	2.02	-1.28
362S162-43	3.625	1.625	0.0451	0.500	0.0712	0.340	1.16	0.710	0.392	1.45	0.127	0.117	0.611	0.537	0.782	0.000230	0.376	2.11	2.04	-1.30
362S162-33	3.625	1.625	0.0346	0.500	0.0765	0.262	0.891	0.551	0.304	1.45	0.0993	0.0913	0.616	0.537	0.789	0.000105	0.297	2.12	2.05	-1.31

Table I - 2

Gross Section Properties

SSMA Studs

C-Sections With Lips



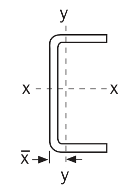
ID	Dimensions							Properties of Full Section													
								Axis x-x			Axis y-y										
	D	B	t	d	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
362S137-68	3.625	1.375	0.0713	0.375	0.1070	0.470	1.60	0.923	0.509	1.40	0.109	0.112	0.481	0.403	0.592	0.000797	0.302	2.06	1.76	-0.959
362S137-54	3.625	1.375	0.0566	0.375	0.0849	0.380	1.29	0.756	0.417	1.41	0.0911	0.0939	0.490	0.405	0.601	0.000405	0.251	2.04	1.79	-0.978
362S137-43	3.625	1.375	0.0451	0.375	0.0712	0.306	1.04	0.616	0.340	1.42	0.0755	0.0779	0.497	0.406	0.608	0.000207	0.208	2.03	1.80	-0.991
362S137-33	3.625	1.375	0.0346	0.375	0.0765	0.236	0.803	0.479	0.264	1.42	0.0594	0.0612	0.501	0.405	0.615	0.0000942	0.165	2.03	1.81	-1.00
350S162-68	3.500	1.625	0.0713	0.500	0.1070	0.515	1.75	0.985	0.563	1.38	0.184	0.170	0.597	0.544	0.772	0.000872	0.514	2.07	1.98	-1.28
350S162-54	3.500	1.625	0.0566	0.500	0.0849	0.415	1.41	0.804	0.460	1.39	0.152	0.141	0.606	0.545	0.782	0.000443	0.426	2.07	2.00	-1.30
350S162-43	3.500	1.625	0.0451	0.500	0.0712	0.334	1.14	0.655	0.374	1.40	0.125	0.116	0.612	0.546	0.789	0.000227	0.350	2.07	2.01	-1.31
350S162-33	3.500	1.625	0.0346	0.500	0.0765	0.258	0.876	0.508	0.291	1.40	0.0981	0.0909	0.617	0.546	0.796	0.000103	0.277	2.07	2.03	-1.32
250S162-68	2.500	1.625	0.0713	0.500	0.1070	0.444	1.51	0.450	0.360	1.01	0.162	0.162	0.605	0.625	0.835	0.000752	0.268	1.81	1.85	-1.42
250S162-54	2.500	1.625	0.0566	0.500	0.0849	0.358	1.22	0.370	0.296	1.02	0.135	0.135	0.613	0.627	0.845	0.000383	0.223	1.81	1.87	-1.44
250S162-43	2.500	1.625	0.0451	0.500	0.0712	0.289	0.983	0.302	0.242	1.02	0.111	0.111	0.620	0.628	0.852	0.000196	0.184	1.82	1.89	-1.46
250S162-33	2.500	1.625	0.0346	0.500	0.0765	0.223	0.759	0.235	0.188	1.03	0.0870	0.0872	0.624	0.628	0.859	0.0000891	0.146	1.83	1.90	-1.47
250S137-68	2.500	1.375	0.0713	0.375	0.1070	0.390	1.33	0.386	0.309	0.995	0.0956	0.107	0.495	0.479	0.653	0.000661	0.138	1.61	1.56	-1.10
250S137-54	2.500	1.375	0.0566	0.375	0.0849	0.316	1.07	0.318	0.255	1.00	0.0802	0.0897	0.504	0.481	0.663	0.000337	0.115	1.61	1.58	-1.12
250S137-43	2.500	1.375	0.0451	0.375	0.0712	0.255	0.868	0.261	0.209	1.01	0.0665	0.0745	0.511	0.482	0.670	0.000173	0.0959	1.62	1.60	-1.13
250S137-33	2.500	1.375	0.0346	0.375	0.0765	0.197	0.671	0.203	0.163	1.02	0.0524	0.0586	0.515	0.482	0.677	0.0000787	0.0764	1.63	1.61	-1.14

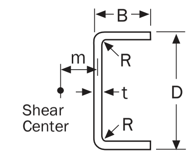
Table I - 3

Gross Section Properties

SSMA Tracks

C-Sections Without Lips





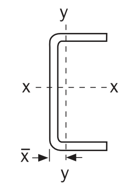
ID	Dimensions						Properties of Full Section													
							Axis x-x			Axis y-y										
	D	B	t	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
1200T200-97	12.356	2.000	0.1017	0.1525	1.63	5.53	29.8	4.82	4.28	0.410	0.240	0.502	0.289	0.476	0.00560	11.9	10.1	4.37	-0.714
1200T200-68	12.250	2.000	0.0713	0.1069	1.14	3.88	20.8	3.39	4.27	0.294	0.171	0.508	0.277	0.483	0.00193	8.43	9.92	4.36	-0.725
1200T200-54*	12.198	2.000	0.0566	0.0849	0.905	3.08	16.5	2.70	4.27	0.236	0.136	0.510	0.272	0.487	0.000966	6.71	9.85	4.36	-0.730
1200T150-97	12.356	1.500	0.1017	0.1525	1.52	5.18	26.0	4.21	4.13	0.176	0.135	0.340	0.191	0.301	0.00525	5.33	13.0	4.17	-0.441
1200T150-68	12.250	1.500	0.0713	0.1069	1.07	3.63	18.1	2.96	4.12	0.127	0.0964	0.345	0.179	0.307	0.00181	3.79	12.7	4.16	-0.450
1200T150-54*	12.198	1.500	0.0566	0.0849	0.848	2.88	14.4	2.36	4.12	0.103	0.0773	0.348	0.173	0.310	0.000906	3.03	12.6	4.16	-0.454
1200T125-97	12.356	1.250	0.1017	0.1525	1.47	5.01	24.1	3.90	4.04	0.102	0.0931	0.264	0.151	0.222	0.00508	3.17	15.5	4.07	-0.322
1200T125-68	12.250	1.250	0.0713	0.1069	1.03	3.51	16.8	2.75	4.04	0.0744	0.0669	0.268	0.138	0.227	0.00175	2.27	15.1	4.06	-0.329
1200T125-54*	12.198	1.250	0.0566	0.0849	0.820	2.79	13.3	2.19	4.03	0.0601	0.0537	0.271	0.131	0.230	0.000876	1.82	15.0	4.06	-0.333
1000T200-97	10.356	2.000	0.1017	0.1525	1.42	4.83	19.1	3.69	3.66	0.397	0.237	0.528	0.323	0.519	0.00490	7.92	7.36	3.79	-0.791
1000T200-68	10.250	2.000	0.0713	0.1069	0.997	3.39	13.3	2.59	3.65	0.284	0.168	0.534	0.312	0.527	0.00169	5.58	7.25	3.78	-0.803
1000T200-54	10.198	2.000	0.0566	0.0849	0.792	2.69	10.5	2.06	3.65	0.228	0.135	0.537	0.306	0.531	0.000845	4.43	7.20	3.77	-0.809
1000T200-43*	10.161	2.000	0.0451	0.0712	0.631	2.15	8.36	1.65	3.64	0.183	0.108	0.539	0.302	0.534	0.000428	3.54	7.16	3.77	-0.813
1000T150-97	10.356	1.500	0.1017	0.1525	1.32	4.49	16.4	3.17	3.53	0.172	0.133	0.361	0.213	0.332	0.00455	3.56	9.26	3.58	-0.495
1000T150-68	10.250	1.500	0.0713	0.1069	0.926	3.15	11.4	2.23	3.52	0.124	0.0954	0.366	0.201	0.339	0.00157	2.52	9.09	3.57	-0.505
1000T150-54	10.198	1.500	0.0566	0.0849	0.735	2.50	9.06	1.78	3.51	0.0998	0.0765	0.368	0.195	0.342	0.000785	2.01	9.00	3.57	-0.509
1000T150-43*	10.161	1.500	0.0451	0.0712	0.586	1.99	7.21	1.42	3.51	0.0804	0.0614	0.370	0.191	0.345	0.000397	1.61	8.94	3.56	-0.513
1000T125-97	10.356	1.250	0.1017	0.1525	1.27	4.32	15.1	2.91	3.45	0.100	0.0923	0.281	0.167	0.247	0.00438	2.12	10.9	3.48	-0.363
1000T125-68	10.250	1.250	0.0713	0.1069	0.890	3.03	10.5	2.05	3.44	0.0727	0.0663	0.286	0.154	0.253	0.00151	1.51	10.7	3.47	-0.372
1000T125-54	10.198	1.250	0.0566	0.0849	0.707	2.40	8.33	1.63	3.43	0.0587	0.0533	0.288	0.148	0.256	0.000755	1.21	10.6	3.47	-0.376
1000T125-43*	10.161	1.250	0.0451	0.0712	0.563	1.92	6.63	1.31	3.43	0.0474	0.0428	0.290	0.143	0.259	0.000382	0.973	10.5	3.46	-0.379
800T200-97	8.356	2.000	0.1017	0.1525	1.22	4.14	11.2	2.68	3.03	0.379	0.232	0.558	0.369	0.571	0.00420	4.79	5.18	3.21	-0.889
800T200-68	8.250	2.000	0.0713	0.1069	0.855	2.91	7.79	1.89	3.02	0.272	0.165	0.564	0.358	0.580	0.00145	3.36	5.11	3.20	-0.902
800T200-54	8.198	2.000	0.0566	0.0849	0.679	2.31	6.15	1.50	3.01	0.218	0.132	0.567	0.353	0.584	0.000725	2.66	5.07	3.20	-0.908
800T200-43	8.161	2.000	0.0451	0.0712	0.541	1.84	4.89	1.20	3.01	0.175	0.106	0.569	0.349	0.587	0.000367	2.12	5.04	3.19	-0.913
800T200-33*	8.146	2.000	0.0346	0.0764	0.415	1.41	3.75	0.921	3.01	0.135	0.0817	0.571	0.345	0.589	0.000166	1.64	5.03	3.19	-0.917

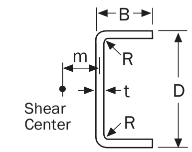
Table I - 3

Gross Section Properties

SSMA Tracks

C-Sections Without Lips





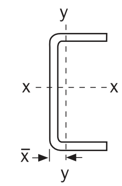
ID	Dimensions						Properties of Full Section												
							Axis x-x			Axis y-y									
	D	B	t	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
800T150-97	8.356	1.500	0.1017	0.1525	1.12	3.80	9.48	2.27	2.91	0.165	0.132	0.385	0.243	0.372	0.00385	2.16	6.26	2.99	-0.564
800T150-68	8.250	1.500	0.0713	0.1069	0.783	2.66	6.59	1.60	2.90	0.119	0.0941	0.390	0.231	0.379	0.00133	1.53	6.13	2.98	-0.575
800T150-54	8.198	1.500	0.0566	0.0849	0.622	2.11	5.21	1.27	2.90	0.0961	0.0754	0.393	0.226	0.383	0.000664	1.22	6.07	2.98	-0.580
800T150-43	8.161	1.500	0.0451	0.0712	0.496	1.69	4.14	1.02	2.89	0.0774	0.0605	0.395	0.221	0.386	0.000336	0.972	6.03	2.98	-0.584
800T150-33*	8.146	1.500	0.0346	0.0764	0.381	1.29	3.18	0.781	2.89	0.0600	0.0467	0.397	0.217	0.388	0.000152	0.751	6.01	2.98	-0.588
800T125-97	8.356	1.250	0.1017	0.1525	1.07	3.62	8.61	2.06	2.84	0.0967	0.0911	0.301	0.189	0.279	0.00367	1.30	7.26	2.89	-0.417
800T125-68	8.250	1.250	0.0713	0.1069	0.748	2.54	6.00	1.45	2.83	0.0703	0.0655	0.307	0.177	0.286	0.00127	0.920	7.10	2.88	-0.427
800T125-54	8.198	1.250	0.0566	0.0849	0.594	2.02	4.75	1.16	2.83	0.0568	0.0526	0.309	0.171	0.289	0.000634	0.735	7.02	2.88	-0.432
800T125-43	8.161	1.250	0.0451	0.0712	0.473	1.61	3.77	0.925	2.82	0.0458	0.0423	0.311	0.166	0.292	0.000321	0.589	6.96	2.87	-0.436
800T125-33*	8.146	1.250	0.0346	0.0764	0.363	1.24	2.90	0.711	2.82	0.0356	0.0327	0.313	0.162	0.294	0.000145	0.456	6.93	2.88	-0.439
600T200-97	6.356	2.000	0.1017	0.1525	1.01	3.45	5.77	1.82	2.39	0.355	0.226	0.591	0.432	0.635	0.00350	2.51	3.53	2.66	-1.02
600T200-68	6.250	2.000	0.0713	0.1069	0.712	2.42	3.99	1.28	2.37	0.254	0.161	0.597	0.422	0.644	0.00121	1.75	3.48	2.65	-1.03
600T200-54	6.198	2.000	0.0566	0.0849	0.565	1.92	3.14	1.01	2.36	0.204	0.129	0.600	0.418	0.649	0.000604	1.38	3.46	2.65	-1.04
600T200-43	6.161	2.000	0.0451	0.0712	0.451	1.53	2.49	0.810	2.35	0.163	0.103	0.602	0.414	0.652	0.000306	1.10	3.45	2.64	-1.04
600T200-33	6.146	2.000	0.0346	0.0764	0.346	1.18	1.91	0.623	2.35	0.126	0.0794	0.604	0.411	0.655	0.000138	0.847	3.44	2.65	-1.05
600T150-97	6.356	1.500	0.1017	0.1525	0.913	3.10	4.78	1.50	2.29	0.156	0.129	0.414	0.285	0.421	0.00315	1.14	3.95	2.42	-0.656
600T150-68	6.250	1.500	0.0713	0.1069	0.641	2.18	3.31	1.06	2.27	0.113	0.0920	0.419	0.275	0.430	0.00109	0.797	3.87	2.41	-0.669
600T150-54	6.198	1.500	0.0566	0.0849	0.509	1.73	2.61	0.843	2.27	0.0907	0.0737	0.422	0.269	0.434	0.000543	0.632	3.84	2.40	-0.675
600T150-43	6.161	1.500	0.0451	0.0712	0.406	1.38	2.07	0.673	2.26	0.0730	0.0592	0.424	0.265	0.437	0.000275	0.504	3.81	2.40	-0.680
600T150-33	6.146	1.500	0.0346	0.0764	0.311	1.06	1.59	0.517	2.26	0.0566	0.0457	0.426	0.262	0.439	0.000124	0.390	3.80	2.40	-0.684
600T150-30	6.141	1.500	0.0312	0.0781	0.281	0.954	1.43	0.467	2.26	0.0512	0.0413	0.427	0.261	0.440	0.0000911	0.352	3.80	2.40	-0.685
600T150-27*	6.136	1.500	0.0283	0.0796	0.255	0.866	1.30	0.424	2.26	0.0465	0.0375	0.428	0.260	0.441	0.0000680	0.320	3.79	2.40	-0.686
600T125-97	6.356	1.250	0.1017	0.1525	0.862	2.93	4.28	1.35	2.23	0.0919	0.0894	0.327	0.221	0.321	0.00297	0.685	4.43	2.31	-0.491
600T125-68	6.250	1.250	0.0713	0.1069	0.605	2.06	2.97	0.950	2.22	0.0668	0.0642	0.332	0.210	0.329	0.00103	0.483	4.33	2.30	-0.503
600T125-54	6.198	1.250	0.0566	0.0849	0.480	1.63	2.34	0.756	2.21	0.0539	0.0516	0.335	0.204	0.332	0.000513	0.384	4.28	2.29	-0.508
600T125-43	6.161	1.250	0.0451	0.0712	0.383	1.30	1.86	0.604	2.21	0.0435	0.0415	0.337	0.200	0.335	0.000260	0.307	4.24	2.29	-0.513
600T125-33	6.146	1.250	0.0346	0.0764	0.294	0.999	1.43	0.465	2.20	0.0338	0.0321	0.339	0.196	0.337	0.000117	0.238	4.22	2.29	-0.516
600T125-30	6.141	1.250	0.0312	0.0781	0.265	0.901	1.29	0.420	2.20	0.0306	0.0290	0.340	0.195	0.338	0.0000860	0.215	4.22	2.29	-0.518
600T125-27*	6.136	1.250	0.0283	0.0796	0.241	0.818	1.17	0.381	2.20	0.0278	0.0264	0.340	0.194	0.339	0.0000642	0.196	4.21	2.29	-0.519

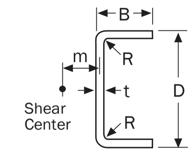
Table I - 3

Gross Section Properties

SSMA Tracks

C-Sections Without Lips





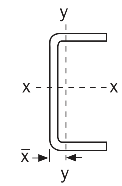
ID	Dimensions						Properties of Full Section												
							Axis x-x			Axis y-y									
	D	B	t	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
550T200-68	5.750	2.000	0.0713	0.1069	0.676	2.30	3.27	1.14	2.20	0.248	0.159	0.606	0.443	0.663	0.00115	1.43	3.16	2.52	-1.07
550T200-54	5.698	2.000	0.0566	0.0849	0.537	1.83	2.58	0.905	2.19	0.199	0.127	0.609	0.438	0.668	0.000573	1.13	3.14	2.52	-1.08
550T200-43	5.661	2.000	0.0451	0.0712	0.428	1.46	2.04	0.722	2.19	0.160	0.102	0.611	0.435	0.671	0.000290	0.900	3.13	2.51	-1.08
550T200-33	5.646	2.000	0.0346	0.0764	0.329	1.12	1.57	0.555	2.18	0.123	0.0787	0.613	0.431	0.674	0.000131	0.694	3.12	2.52	-1.09
550T150-68	5.750	1.500	0.0713	0.1069	0.605	2.06	2.70	0.939	2.11	0.111	0.0912	0.427	0.289	0.445	0.00103	0.655	3.42	2.27	-0.698
550T150-54	5.698	1.500	0.0566	0.0849	0.480	1.63	2.13	0.747	2.11	0.0889	0.0731	0.430	0.284	0.449	0.000513	0.519	3.39	2.26	-0.704
550T150-43	5.661	1.500	0.0451	0.0712	0.383	1.30	1.69	0.596	2.10	0.0716	0.0587	0.433	0.280	0.452	0.000260	0.414	3.36	2.26	-0.709
550T150-33	5.646	1.500	0.0346	0.0764	0.294	0.999	1.29	0.459	2.10	0.0555	0.0453	0.434	0.276	0.455	0.000117	0.320	3.35	2.26	-0.714
550T150-30	5.641	1.500	0.0312	0.0781	0.265	0.901	1.17	0.414	2.10	0.0502	0.0410	0.435	0.275	0.455	0.0000860	0.289	3.35	2.26	-0.715
550T150-27	5.636	1.500	0.0283	0.0796	0.241	0.818	1.06	0.376	2.10	0.0456	0.0372	0.436	0.274	0.456	0.0000642	0.263	3.35	2.26	-0.716
550T125-68	5.750	1.250	0.0713	0.1069	0.569	1.94	2.41	0.839	2.06	0.0656	0.0638	0.340	0.221	0.341	0.000965	0.397	3.77	2.15	-0.526
550T125-54	5.698	1.250	0.0566	0.0849	0.452	1.54	1.90	0.668	2.05	0.0530	0.0512	0.342	0.215	0.345	0.000483	0.315	3.72	2.15	-0.532
550T125-43	5.661	1.250	0.0451	0.0712	0.360	1.23	1.51	0.533	2.05	0.0428	0.0412	0.345	0.211	0.348	0.000244	0.252	3.69	2.14	-0.537
550T125-33	5.646	1.250	0.0346	0.0764	0.277	0.941	1.16	0.410	2.05	0.0332	0.0318	0.346	0.207	0.350	0.000110	0.195	3.68	2.15	-0.541
550T125-30	5.641	1.250	0.0312	0.0781	0.250	0.848	1.04	0.371	2.05	0.0301	0.0288	0.347	0.206	0.351	0.0000810	0.176	3.67	2.15	-0.542
550T125-27	5.636	1.250	0.0283	0.0796	0.226	0.770	0.948	0.336	2.05	0.0273	0.0262	0.348	0.205	0.352	0.0000604	0.160	3.67	2.15	-0.543
400T200-68	4.250	2.000	0.0713	0.1069	0.569	1.94	1.62	0.761	1.69	0.227	0.153	0.632	0.519	0.725	0.000965	0.702	2.39	2.17	-1.21
400T200-54	4.198	2.000	0.0566	0.0849	0.452	1.54	1.27	0.604	1.68	0.182	0.123	0.635	0.515	0.730	0.000483	0.551	2.38	2.17	-1.22
400T200-43	4.161	2.000	0.0451	0.0712	0.360	1.23	1.00	0.482	1.67	0.146	0.0982	0.637	0.512	0.734	0.000244	0.436	2.37	2.16	-1.22
400T200-33	4.146	2.000	0.0346	0.0764	0.277	0.941	0.768	0.371	1.67	0.113	0.0757	0.639	0.509	0.737	0.000110	0.336	2.37	2.17	-1.23
400T150-68	4.250	1.500	0.0713	0.1069	0.498	1.69	1.31	0.615	1.62	0.102	0.0883	0.453	0.343	0.496	0.000844	0.320	2.31	1.86	-0.804
400T150-54	4.198	1.500	0.0566	0.0849	0.396	1.35	1.03	0.489	1.61	0.0822	0.0708	0.456	0.339	0.501	0.000422	0.252	2.30	1.86	-0.811
400T150-43	4.161	1.500	0.0451	0.0712	0.315	1.07	0.811	0.390	1.60	0.0662	0.0568	0.458	0.335	0.504	0.000214	0.200	2.29	1.86	-0.817
400T150-33	4.146	1.500	0.0346	0.0764	0.242	0.823	0.622	0.300	1.60	0.0513	0.0439	0.460	0.332	0.507	0.0000966	0.155	2.28	1.86	-0.821
400T150-30	4.141	1.500	0.0312	0.0781	0.218	0.742	0.561	0.271	1.60	0.0464	0.0396	0.461	0.331	0.508	0.0000708	0.140	2.28	1.86	-0.823
400T150-27	4.136	1.500	0.0283	0.0796	0.198	0.673	0.509	0.246	1.60	0.0422	0.0360	0.461	0.330	0.509	0.0000529	0.127	2.28	1.86	-0.824

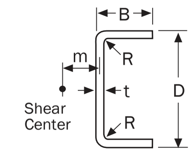
Table I - 3

Gross Section Properties

SSMA Tracks

C-Sections Without Lips





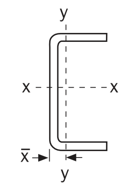
ID	Dimensions						Properties of Full Section												
							Axis x-x			Axis y-y									
	D	B	t	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
400T125-68	4.250	1.250	0.0713	0.1069	0.462	1.57	1.15	0.541	1.58	0.0611	0.0620	0.364	0.264	0.386	0.000783	0.194	2.40	1.73	-0.614
400T125-54	4.198	1.250	0.0566	0.0849	0.367	1.25	0.904	0.431	1.57	0.0493	0.0498	0.367	0.259	0.390	0.000392	0.154	2.37	1.73	-0.621
400T125-43	4.161	1.250	0.0451	0.0712	0.293	0.995	0.716	0.344	1.56	0.0398	0.0400	0.369	0.255	0.394	0.000198	0.122	2.36	1.72	-0.626
400T125-33	4.146	1.250	0.0346	0.0764	0.225	0.764	0.549	0.265	1.56	0.0309	0.0309	0.371	0.251	0.396	0.0000897	0.0946	2.35	1.73	-0.630
400T125-30	4.141	1.250	0.0312	0.0781	0.203	0.689	0.495	0.239	1.56	0.0280	0.0280	0.371	0.250	0.397	0.0000658	0.0855	2.35	1.73	-0.632
400T125-27	4.136	1.250	0.0283	0.0796	0.184	0.625	0.449	0.217	1.56	0.0254	0.0254	0.372	0.249	0.398	0.0000491	0.0777	2.35	1.73	-0.633
400T125-18*	4.122	1.250	0.0188	0.0843	0.122	0.416	0.298	0.145	1.56	0.0171	0.0170	0.374	0.246	0.400	0.0000144	0.0520	2.34	1.73	-0.637
362T200-68	3.875	2.000	0.0713	0.1069	0.543	1.85	1.31	0.675	1.55	0.221	0.151	0.638	0.543	0.743	0.000919	0.564	2.24	2.09	-1.25
362T200-54	3.823	2.000	0.0566	0.0849	0.431	1.47	1.02	0.536	1.54	0.177	0.121	0.641	0.539	0.748	0.000460	0.442	2.24	2.09	-1.26
362T200-43	3.786	2.000	0.0451	0.0712	0.344	1.17	0.808	0.427	1.53	0.142	0.0969	0.643	0.536	0.752	0.000233	0.350	2.23	2.09	-1.27
362T200-33	3.771	2.000	0.0346	0.0764	0.264	0.897	0.619	0.329	1.53	0.110	0.0747	0.645	0.533	0.754	0.000105	0.269	2.23	2.09	-1.27
362T150-68	3.875	1.500	0.0713	0.1069	0.471	1.60	1.05	0.542	1.49	0.0995	0.0873	0.460	0.361	0.511	0.000799	0.257	2.10	1.77	-0.836
362T150-54	3.823	1.500	0.0566	0.0849	0.374	1.27	0.823	0.431	1.48	0.0801	0.0700	0.463	0.356	0.516	0.000400	0.202	2.09	1.77	-0.844
362T150-43	3.786	1.500	0.0451	0.0712	0.298	1.01	0.650	0.344	1.48	0.0644	0.0562	0.465	0.353	0.519	0.000202	0.160	2.08	1.77	-0.850
362T150-33	3.771	1.500	0.0346	0.0764	0.229	0.779	0.499	0.264	1.48	0.0499	0.0434	0.467	0.350	0.522	0.0000914	0.124	2.08	1.77	-0.854
362T150-30	3.766	1.500	0.0312	0.0781	0.207	0.703	0.449	0.239	1.48	0.0451	0.0392	0.467	0.348	0.523	0.0000671	0.112	2.07	1.77	-0.856
362T150-27	3.761	1.500	0.0283	0.0796	0.188	0.637	0.408	0.217	1.48	0.0410	0.0356	0.468	0.348	0.524	0.0000501	0.102	2.07	1.77	-0.857
362T125-68	3.875	1.250	0.0713	0.1069	0.436	1.48	0.921	0.475	1.45	0.0597	0.0613	0.370	0.277	0.399	0.000738	0.156	2.13	1.63	-0.641
362T125-54	3.823	1.250	0.0566	0.0849	0.346	1.18	0.723	0.378	1.45	0.0481	0.0493	0.373	0.273	0.404	0.000369	0.123	2.11	1.63	-0.648
362T125-43	3.786	1.250	0.0451	0.0712	0.276	0.938	0.571	0.302	1.44	0.0388	0.0396	0.375	0.269	0.407	0.000187	0.0978	2.10	1.63	-0.654
362T125-33	3.771	1.250	0.0346	0.0764	0.212	0.720	0.438	0.232	1.44	0.0301	0.0306	0.377	0.266	0.409	0.0000845	0.0756	2.09	1.63	-0.658
362T125-30	3.766	1.250	0.0312	0.0781	0.191	0.649	0.395	0.210	1.44	0.0273	0.0277	0.378	0.265	0.410	0.0000620	0.0684	2.09	1.63	-0.659
362T125-27	3.761	1.250	0.0283	0.0796	0.173	0.589	0.358	0.191	1.44	0.0248	0.0252	0.378	0.264	0.411	0.0000463	0.0622	2.09	1.63	-0.661
362T125-18	3.747	1.250	0.0188	0.0843	0.115	0.392	0.238	0.127	1.44	0.0167	0.0168	0.380	0.261	0.413	0.0000136	0.0416	2.08	1.63	-0.665
350T200-68	3.750	2.000	0.0713	0.1069	0.534	1.81	1.21	0.647	1.51	0.218	0.151	0.640	0.552	0.749	0.000904	0.522	2.20	2.07	-1.26
350T200-54	3.698	2.000	0.0566	0.0849	0.424	1.44	0.949	0.513	1.50	0.175	0.120	0.642	0.548	0.754	0.000453	0.409	2.19	2.07	-1.27
350T200-43	3.661	2.000	0.0451	0.0712	0.338	1.15	0.749	0.409	1.49	0.140	0.0965	0.645	0.545	0.758	0.000229	0.323	2.19	2.07	-1.28
350T200-33	3.646	2.000	0.0346	0.0764	0.259	0.882	0.574	0.315	1.49	0.108	0.0744	0.647	0.542	0.761	0.000104	0.249	2.19	2.07	-1.29

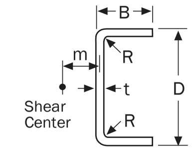
Table I - 3

Gross Section Properties

SSMA Tracks

C-Sections Without Lips





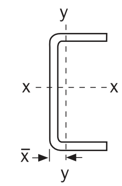
ID	Dimensions						Properties of Full Section												
							Axis x-x			Axis y-y									
	D	B	t	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
350T150-68	3.750	1.500	0.0713	0.1069	0.462	1.57	0.972	0.518	1.45	0.0986	0.0870	0.462	0.367	0.516	0.000783	0.238	2.04	1.74	-0.847
350T150-54	3.698	1.500	0.0566	0.0849	0.367	1.25	0.761	0.412	1.44	0.0793	0.0697	0.465	0.362	0.521	0.000392	0.187	2.02	1.74	-0.855
350T150-43	3.661	1.500	0.0451	0.0712	0.293	0.995	0.601	0.329	1.43	0.0638	0.0559	0.467	0.359	0.525	0.000198	0.148	2.01	1.74	-0.861
350T150-33	3.646	1.500	0.0346	0.0764	0.225	0.764	0.461	0.253	1.43	0.0494	0.0432	0.469	0.356	0.527	0.0000897	0.114	2.01	1.74	-0.866
350T150-30	3.641	1.500	0.0312	0.0781	0.203	0.689	0.416	0.228	1.43	0.0447	0.0390	0.470	0.355	0.528	0.0000658	0.103	2.01	1.74	-0.867
350T150-27	3.636	1.500	0.0283	0.0796	0.184	0.625	0.377	0.207	1.43	0.0406	0.0355	0.470	0.354	0.529	0.0000491	0.0939	2.01	1.74	-0.869
350T125-68	3.750	1.250	0.0713	0.1069	0.427	1.45	0.851	0.454	1.41	0.0591	0.0611	0.372	0.283	0.403	0.000723	0.144	2.05	1.60	-0.650
350T125-54	3.698	1.250	0.0566	0.0849	0.339	1.15	0.668	0.361	1.40	0.0477	0.0491	0.375	0.278	0.408	0.000362	0.114	2.03	1.60	-0.658
350T125-43	3.661	1.250	0.0451	0.0712	0.270	0.919	0.528	0.288	1.40	0.0385	0.0394	0.377	0.274	0.412	0.000183	0.0904	2.02	1.59	-0.663
350T125-33	3.646	1.250	0.0346	0.0764	0.208	0.705	0.405	0.222	1.40	0.0299	0.0305	0.379	0.271	0.414	0.0000828	0.0699	2.01	1.59	-0.668
350T125-30	3.641	1.250	0.0312	0.0781	0.187	0.636	0.365	0.200	1.40	0.0270	0.0276	0.380	0.270	0.415	0.0000607	0.0632	2.01	1.59	-0.669
350T125-27	3.636	1.250	0.0283	0.0796	0.170	0.577	0.331	0.182	1.40	0.0246	0.0251	0.381	0.269	0.416	0.0000453	0.0574	2.01	1.60	-0.670
350T125-18	3.622	1.250	0.0188	0.0843	0.113	0.384	0.220	0.121	1.40	0.0165	0.0168	0.382	0.266	0.418	0.0000133	0.0384	2.00	1.60	-0.675
250T200-68	2.750	2.000	0.0713	0.1069	0.462	1.57	0.600	0.437	1.14	0.196	0.143	0.652	0.631	0.800	0.000783	0.251	1.92	1.92	-1.40
250T200-54	2.698	2.000	0.0566	0.0849	0.367	1.25	0.466	0.346	1.13	0.157	0.115	0.654	0.628	0.806	0.000392	0.195	1.92	1.92	-1.41
250T200-43	2.661	2.000	0.0451	0.0712	0.293	0.995	0.366	0.275	1.12	0.126	0.0918	0.657	0.625	0.810	0.000198	0.153	1.92	1.92	-1.41
250T200-33	2.646	2.000	0.0346	0.0764	0.225	0.764	0.280	0.212	1.12	0.0974	0.0707	0.658	0.623	0.813	0.0000897	0.118	1.93	1.92	-1.42
250T150-68	2.750	1.500	0.0713	0.1069	0.391	1.33	0.472	0.344	1.10	0.0893	0.0833	0.478	0.427	0.561	0.000663	0.114	1.61	1.53	-0.953
250T150-54	2.698	1.500	0.0566	0.0849	0.311	1.06	0.368	0.273	1.09	0.0718	0.0667	0.481	0.423	0.566	0.000332	0.0887	1.61	1.53	-0.961
250T150-43	2.661	1.500	0.0451	0.0712	0.248	0.842	0.289	0.217	1.08	0.0578	0.0535	0.483	0.420	0.570	0.000168	0.0698	1.61	1.53	-0.968
250T150-33	2.646	1.500	0.0346	0.0764	0.190	0.647	0.221	0.167	1.08	0.0447	0.0413	0.485	0.418	0.573	0.0000759	0.0539	1.61	1.53	-0.973
250T150-30	2.641	1.500	0.0312	0.0781	0.172	0.583	0.200	0.151	1.08	0.0404	0.0373	0.486	0.417	0.574	0.0000557	0.0486	1.61	1.53	-0.975
250T150-27	2.636	1.500	0.0283	0.0796	0.156	0.529	0.181	0.137	1.08	0.0368	0.0339	0.486	0.416	0.575	0.0000416	0.0442	1.61	1.53	-0.976
250T125-68	2.750	1.250	0.0713	0.1069	0.355	1.21	0.409	0.297	1.07	0.0539	0.0587	0.389	0.332	0.444	0.000602	0.0689	1.51	1.36	-0.740
250T125-54	2.698	1.250	0.0566	0.0849	0.282	0.960	0.318	0.236	1.06	0.0435	0.0471	0.392	0.328	0.449	0.000301	0.0539	1.50	1.36	-0.749
250T125-43	2.661	1.250	0.0451	0.0712	0.225	0.765	0.250	0.188	1.06	0.0351	0.0379	0.395	0.325	0.453	0.000153	0.0425	1.49	1.36	-0.755
250T125-33	2.646	1.250	0.0346	0.0764	0.173	0.588	0.192	0.145	1.05	0.0272	0.0293	0.397	0.322	0.456	0.0000690	0.0328	1.49	1.36	-0.760
250T125-30	2.641	1.250	0.0312	0.0781	0.156	0.530	0.173	0.131	1.05	0.0246	0.0265	0.397	0.321	0.456	0.0000506	0.0297	1.49	1.36	-0.762
250T125-27	2.636	1.250	0.0283	0.0796	0.142	0.481	0.157	0.119	1.05	0.0224	0.0241	0.398	0.320	0.457	0.0000378	0.0270	1.49	1.36	-0.763
250T125-18	2.622	1.250	0.0188	0.0843	0.0941	0.320	0.104	0.0794	1.05	0.0150	0.0161	0.400	0.317	0.460	0.0000111	0.0180	1.49	1.36	-0.767

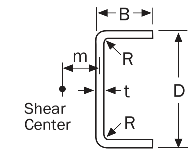
Table I - 3

Gross Section Properties

SSMA Tracks

C-Sections Without Lips



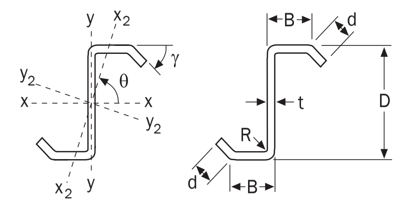


ID	Dimensions						Properties of Full Section												
							Axis x-x			Axis y-y									
	D	B	t	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
162T125-33	1.771	1.250	0.0346	0.0764	0.143	0.485	0.0772	0.0872	0.736	0.0238	0.0275	0.408	0.386	0.499	0.0000569	0.0128	1.22	1.21	-0.868
162T125-30	1.766	1.250	0.0312	0.0781	0.129	0.437	0.0695	0.0788	0.735	0.0215	0.0249	0.409	0.386	0.500	0.0000417	0.0115	1.22	1.21	-0.870
162T125-27	1.761	1.250	0.0283	0.0796	0.117	0.397	0.0630	0.0716	0.735	0.0196	0.0226	0.410	0.385	0.501	0.0000312	0.0105	1.22	1.21	-0.872
162T125-18	1.747	1.250	0.0188	0.0843	0.0776	0.264	0.0417	0.0478	0.733	0.0131	0.0151	0.411	0.382	0.503	0.00000915	0.00699	1.22	1.22	-0.876

Table I - 4

Gross Section Properties

Z-Sections With Lips

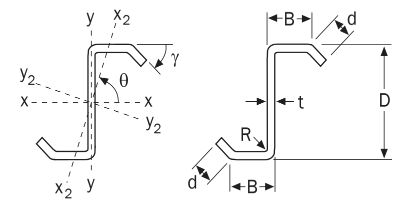


ID	Dimensions								Properties of Full Section												
									Axis x-x			Axis y-y									
	D	B	t	d	γ	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	I_{xy}	I_{x2}	I_{y2}	r_{min}	θ	J	C_w
	in.	in.	in.	in.	deg	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{4}	in.^{4}	in.^{4}	in.	deg	in.^{4}	in.^{6}
12ZS3.25x105	12.000	3.250	0.105	0.990	50	0.1875	2.09	7.12	43.7	7.29	4.57	4.67	1.22	1.49	10.2	2.18	46.2	1.02	76.3	0.00769	123
12ZS3.25x085	12.000	3.250	0.085	0.960	50	0.1875	1.70	5.76	35.5	5.92	4.58	3.75	0.982	1.49	8.21	1.76	37.5	1.02	76.3	0.00408	98.6
12ZS3.25x070	12.000	3.250	0.070	0.930	50	0.1875	1.40	4.75	29.3	4.89	4.58	3.07	0.806	1.48	6.74	1.44	31.0	1.02	76.4	0.00228	80.8
12ZS2.75x105	12.000	2.750	0.105	0.990	50	0.1875	1.99	6.76	40.0	6.67	4.49	3.13	0.938	1.25	7.78	1.55	41.6	0.883	78.6	0.00731	85.1
12ZS2.75x085	12.000	2.750	0.085	0.960	50	0.1875	1.61	5.47	32.5	5.42	4.49	2.51	0.755	1.25	6.28	1.25	33.8	0.880	78.6	0.00388	68.4
12ZS2.75x070	12.000	2.750	0.070	0.930	50	0.1875	1.33	4.51	26.8	4.47	4.50	2.05	0.620	1.24	5.16	1.02	27.9	0.878	78.7	0.00217	56.0
12ZS2.25x105	12.000	2.250	0.105	0.990	50	0.1875	1.88	6.40	36.3	6.05	4.39	1.96	0.692	1.02	5.71	1.03	37.2	0.741	80.8	0.00692	55.5
12ZS2.25x085	12.000	2.250	0.085	0.960	50	0.1875	1.53	5.18	29.5	4.91	4.40	1.57	0.557	1.02	4.61	0.831	30.2	0.738	80.9	0.00367	44.6
12ZS2.25x070	12.000	2.250	0.070	0.930	50	0.1875	1.26	4.27	24.3	4.06	4.40	1.28	0.457	1.01	3.78	0.681	24.9	0.736	80.9	0.00205	36.5
10ZS3.25x105	10.000	3.250	0.105	0.990	50	0.1875	1.88	6.40	28.4	5.69	3.89	4.67	1.22	1.58	8.41	2.00	31.1	1.03	72.4	0.00692	81.8
10ZS3.25x085	10.000	3.250	0.085	0.960	50	0.1875	1.53	5.18	23.1	4.62	3.89	3.75	0.982	1.57	6.79	1.61	25.3	1.03	72.5	0.00367	65.9
10ZS3.25x070	10.000	3.250	0.070	0.930	50	0.1875	1.26	4.27	19.1	3.82	3.90	3.07	0.806	1.56	5.59	1.32	20.9	1.02	72.6	0.00205	54.0
10ZS3.25x065	10.000	3.250	0.065	0.920	50	0.1875	1.17	3.96	17.8	3.55	3.90	2.85	0.747	1.56	5.18	1.22	19.4	1.02	72.6	0.00164	50.1
10ZS3.25x059	10.000	3.250	0.059	0.910	50	0.1875	1.06	3.60	16.1	3.23	3.91	2.58	0.677	1.56	4.70	1.11	17.6	1.02	72.6	0.00123	45.3
10ZS2.75x105	10.000	2.750	0.105	0.990	50	0.1875	1.78	6.05	25.9	5.17	3.81	3.13	0.938	1.33	6.44	1.43	27.6	0.897	75.2	0.00654	57.0
10ZS2.75x085	10.000	2.750	0.085	0.960	50	0.1875	1.44	4.90	21.0	4.21	3.82	2.51	0.755	1.32	5.20	1.15	22.4	0.894	75.4	0.00347	45.8
10ZS2.75x070	10.000	2.750	0.070	0.930	50	0.1875	1.19	4.03	17.4	3.47	3.83	2.05	0.620	1.32	4.27	0.943	18.5	0.892	75.4	0.00194	37.5
10ZS2.75x065	10.000	2.750	0.065	0.920	50	0.1875	1.10	3.74	16.2	3.23	3.83	1.90	0.575	1.31	3.96	0.874	17.2	0.891	75.5	0.00155	34.8
10ZS2.75x059	10.000	2.750	0.059	0.910	50	0.1875	0.999	3.40	14.7	2.94	3.83	1.72	0.521	1.31	3.59	0.792	15.6	0.890	75.5	0.00116	31.5
10ZS2.25x105	10.000	2.250	0.105	0.990	50	0.1875	1.67	5.69	23.3	4.66	3.73	1.96	0.692	1.08	4.72	0.962	24.3	0.758	78.1	0.00615	37.3
10ZS2.25x085	10.000	2.250	0.085	0.960	50	0.1875	1.36	4.61	18.9	3.79	3.74	1.57	0.557	1.08	3.81	0.773	19.7	0.756	78.2	0.00326	30.0
10ZS2.25x070	10.000	2.250	0.070	0.930	50	0.1875	1.12	3.79	15.6	3.13	3.75	1.28	0.457	1.07	3.13	0.633	16.3	0.753	78.2	0.00182	24.5
10ZS2.25x065	10.000	2.250	0.065	0.920	50	0.1875	1.04	3.52	14.5	2.91	3.75	1.19	0.424	1.07	2.90	0.587	15.2	0.753	78.3	0.00146	22.8
10ZS2.25x059	10.000	2.250	0.059	0.910	50	0.1875	0.940	3.20	13.2	2.64	3.75	1.08	0.384	1.07	2.63	0.531	13.8	0.752	78.3	0.00109	20.6

Table I - 4

Gross Section Properties

Z-Sections With Lips

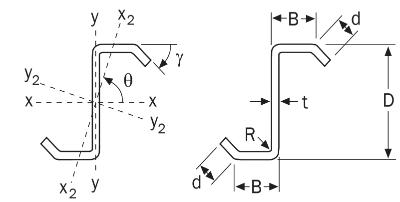


ID	Dimensions								Properties of Full Section												
									Axis x-x			Axis y-y									
	D	B	t	d	γ	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	I_{xy}	I_{x2}	I_{y2}	r_{min}	θ	J	C_w
	in.	in.	in.	in.	deg	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{4}	in.^{4}	in.^{4}	in.	deg	in.^{4}	in.^{6}
9ZS2.25x105	9.000	2.250	0.105	0.990	50	0.1875	1.57	5.33	18.1	4.02	3.39	1.96	0.692	1.12	4.22	0.920	19.1	0.766	76.2	0.00576	29.7
9ZS2.25x085	9.000	2.250	0.085	0.960	50	0.1875	1.27	4.32	14.7	3.27	3.40	1.57	0.557	1.11	3.41	0.739	15.5	0.763	76.3	0.00306	23.8
9ZS2.25x070	9.000	2.250	0.070	0.930	50	0.1875	1.05	3.56	12.2	2.70	3.41	1.28	0.457	1.11	2.80	0.605	12.8	0.761	76.4	0.00171	19.5
9ZS2.25x065	9.000	2.250	0.065	0.920	50	0.1875	0.971	3.30	11.3	2.51	3.41	1.19	0.424	1.11	2.60	0.561	11.9	0.760	76.4	0.00137	18.1
9ZS2.25x059	9.000	2.250	0.059	0.910	50	0.1875	0.881	3.00	10.3	2.28	3.41	1.08	0.384	1.11	2.36	0.508	10.8	0.759	76.4	0.00102	16.4
8ZS3.25x105	8.000	3.250	0.105	0.990	50	0.1875	1.67	5.69	16.9	4.23	3.18	4.67	1.22	1.67	6.66	1.75	19.8	1.02	66.3	0.00615	49.9
8ZS3.25x085	8.000	3.250	0.085	0.960	50	0.1875	1.36	4.61	13.8	3.44	3.19	3.75	0.982	1.66	5.38	1.41	16.1	1.02	66.5	0.00326	40.1
8ZS3.25x070	8.000	3.250	0.070	0.930	50	0.1875	1.12	3.79	11.4	2.85	3.19	3.07	0.806	1.66	4.43	1.15	13.3	1.02	66.6	0.00182	32.9
8ZS3.25x065	8.000	3.250	0.065	0.920	50	0.1875	1.04	3.52	10.6	2.65	3.20	2.85	0.747	1.66	4.11	1.07	12.4	1.02	66.6	0.00146	30.5
8ZS3.25x059	8.000	3.250	0.059	0.910	50	0.1875	0.940	3.20	9.62	2.41	3.20	2.58	0.677	1.66	3.73	0.970	11.2	1.02	66.7	0.00109	27.6
8ZS2.75x105	8.000	2.750	0.105	0.990	50	0.1875	1.57	5.33	15.3	3.82	3.12	3.13	0.938	1.41	5.09	1.28	17.1	0.902	70.0	0.00576	34.9
8ZS2.75x085	8.000	2.750	0.085	0.960	50	0.1875	1.27	4.32	12.4	3.11	3.13	2.51	0.755	1.41	4.11	1.03	13.9	0.899	70.2	0.00306	28.0
8ZS2.75x070	8.000	2.750	0.070	0.930	50	0.1875	1.05	3.56	10.3	2.57	3.14	2.05	0.620	1.40	3.38	0.841	11.5	0.897	70.3	0.00171	23.0
8ZS2.75x065	8.000	2.750	0.065	0.920	50	0.1875	0.971	3.30	9.56	2.39	3.14	1.90	0.575	1.40	3.14	0.780	10.7	0.896	70.3	0.00137	21.3
8ZS2.75x059	8.000	2.750	0.059	0.910	50	0.1875	0.881	3.00	8.69	2.17	3.14	1.72	0.521	1.40	2.85	0.706	9.71	0.895	70.4	0.00102	19.3
8ZS2.25x105	8.000	2.250	0.105	0.990	50	0.1875	1.46	4.98	13.6	3.41	3.05	1.96	0.692	1.16	3.73	0.872	14.7	0.772	73.7	0.00538	22.9
8ZS2.25x085	8.000	2.250	0.085	0.960	50	0.1875	1.19	4.03	11.1	2.77	3.06	1.57	0.557	1.15	3.01	0.700	12.0	0.769	73.9	0.00285	18.4
8ZS2.25x070	8.000	2.250	0.070	0.930	50	0.1875	0.976	3.32	9.18	2.30	3.07	1.28	0.457	1.15	2.47	0.573	9.89	0.767	74.0	0.00159	15.1
8ZS2.25x065	8.000	2.250	0.065	0.920	50	0.1875	0.906	3.08	8.54	2.13	3.07	1.19	0.424	1.15	2.30	0.532	9.20	0.766	74.0	0.00128	14.0
8ZS2.25x059	8.000	2.250	0.059	0.910	50	0.1875	0.822	2.80	7.76	1.94	3.07	1.08	0.384	1.14	2.08	0.481	8.36	0.765	74.0	0.000954	12.7
7ZS2.25x105	7.000	2.250	0.105	0.990	50	0.1875	1.36	4.62	9.92	2.83	2.70	1.96	0.692	1.20	3.23	0.815	11.1	0.774	70.5	0.00499	17.1
7ZS2.25x085	7.000	2.250	0.085	0.960	50	0.1875	1.10	3.74	8.09	2.31	2.71	1.57	0.557	1.20	2.61	0.655	9.01	0.772	70.6	0.00265	13.8
7ZS2.25x070	7.000	2.250	0.070	0.930	50	0.1875	0.906	3.08	6.70	1.91	2.72	1.28	0.457	1.19	2.15	0.536	7.45	0.769	70.8	0.00148	11.3
7ZS2.25x065	7.000	2.250	0.065	0.920	50	0.1875	0.841	2.86	6.23	1.78	2.72	1.19	0.424	1.19	1.99	0.497	6.92	0.769	70.8	0.00118	10.5
7ZS2.25x059	7.000	2.250	0.059	0.910	50	0.1875	0.763	2.60	5.67	1.62	2.72	1.08	0.384	1.19	1.81	0.450	6.29	0.768	70.9	0.000886	9.46

Table I - 4

Gross Section Properties

Z-Sections With Lips

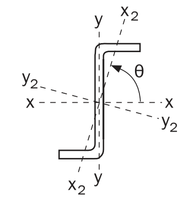


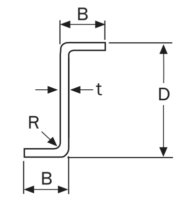
ID	Dimensions								Properties of Full Section													
									Axis x-x			Axis y-y										
	D	B	t	d	γ	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	I_{xy}	I_{x2}	I_{y2}	r_{min}	θ	J	C_w
	in.	in.	in.	in.	deg	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{4}	in.^{4}	in.^{4}	in.	deg	in.^{4}	in.^{6}
6ZS2.25x105	6.000	2.250	0.105	0.990	50	0.1875	1.25	4.26	6.89	2.30	2.35	1.96	0.692	1.25	2.73	0.745	8.11	0.771	66.0	0.00461	12.3
6ZS2.25x085	6.000	2.250	0.085	0.960	50	0.1875	1.02	3.45	5.63	1.88	2.36	1.57	0.557	1.25	2.21	0.599	6.60	0.768	66.3	0.00244	9.85
6ZS2.25x070	6.000	2.250	0.070	0.930	50	0.1875	0.836	2.84	4.66	1.55	2.36	1.28	0.457	1.24	1.82	0.490	5.46	0.766	66.4	0.00136	8.07
6ZS2.25x065	6.000	2.250	0.065	0.920	50	0.1875	0.776	2.64	4.34	1.45	2.37	1.19	0.424	1.24	1.69	0.455	5.08	0.765	66.5	0.00109	7.48
6ZS2.25x059	6.000	2.250	0.059	0.910	50	0.1875	0.704	2.39	3.95	1.32	2.37	1.08	0.384	1.24	1.53	0.412	4.62	0.764	66.6	0.000817	6.77
4ZS2.25x070	4.000	2.250	0.070	0.930	50	0.1875	0.696	2.37	1.82	0.910	1.62	1.28	0.457	1.36	1.17	0.355	2.75	0.714	51.5	0.00114	3.38
4ZS2.25x065	4.000	2.250	0.065	0.920	50	0.1875	0.646	2.20	1.70	0.848	1.62	1.19	0.424	1.36	1.09	0.329	2.56	0.714	51.6	0.000910	3.14
4ZS2.25x059	4.000	2.250	0.059	0.910	50	0.1875	0.586	1.99	1.55	0.773	1.62	1.08	0.384	1.36	0.986	0.298	2.32	0.713	51.7	0.000680	2.84
3.5ZS1.5x070	3.500	1.500	0.070	0.680	50	0.1875	0.521	1.77	0.985	0.563	1.38	0.396	0.208	0.872	0.472	0.134	1.25	0.508	61.0	0.000850	0.830
3.5ZS1.5x065	3.500	1.500	0.065	0.670	50	0.1875	0.483	1.64	0.919	0.525	1.38	0.367	0.193	0.871	0.438	0.125	1.16	0.508	61.1	0.000681	0.769
3.5ZS1.5x059	3.500	1.500	0.059	0.660	50	0.1875	0.439	1.49	0.838	0.479	1.38	0.331	0.175	0.868	0.398	0.113	1.06	0.507	61.2	0.000509	0.695

Table I - 5

Gross Section Properties

Z-Sections Without Lips



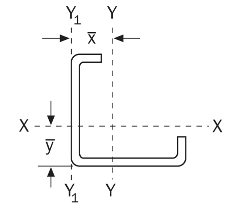


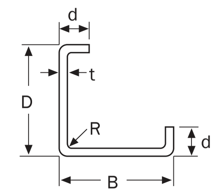
ID	Dimensions						Properties of Full Section												
							Axis x-x			Axis y-y									
	D	B	t	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	I_{xy}	I_{x2}	I_{y2}	r_{min}	θ	J	C_w
	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{4}	in.^{4}	in.^{4}	in.	deg	in.^{4}	in.^{6}
8ZU1.25x105	8.000	1.250	0.105	0.1875	1.06	3.60	7.88	1.97	2.73	0.120	0.100	0.337	0.598	0.0745	7.93	0.265	85.6	0.00389	1.55
8ZU1.25x090	8.000	1.250	0.090	0.1875	0.911	3.10	6.82	1.70	2.74	0.105	0.0872	0.340	0.519	0.0651	6.86	0.267	85.6	0.00246	1.35
8ZU1.25x075	8.000	1.250	0.075	0.1875	0.762	2.59	5.73	1.43	2.74	0.0892	0.0735	0.342	0.439	0.0553	5.77	0.269	85.6	0.00143	1.15
8ZU1.25x060	8.000	1.250	0.060	0.1875	0.612	2.08	4.63	1.16	2.75	0.0727	0.0596	0.345	0.356	0.0450	4.66	0.271	85.6	0.000734	0.943
8ZU1.25x048	8.000	1.250	0.048	0.1875	0.491	1.67	3.73	0.933	2.76	0.0590	0.0481	0.347	0.288	0.0365	3.75	0.273	85.5	0.000377	0.767
6ZU1.25x105	6.000	1.250	0.105	0.1875	0.849	2.89	3.78	1.26	2.11	0.120	0.100	0.376	0.446	0.0667	3.84	0.280	83.2	0.00312	0.818
6ZU1.25x090	6.000	1.250	0.090	0.1875	0.731	2.49	3.28	1.09	2.12	0.105	0.0872	0.379	0.388	0.0583	3.33	0.282	83.1	0.00197	0.718
6ZU1.25x075	6.000	1.250	0.075	0.1875	0.612	2.08	2.77	0.922	2.13	0.0892	0.0735	0.382	0.328	0.0495	2.81	0.285	83.1	0.00115	0.612
6ZU1.25x060	6.000	1.250	0.060	0.1875	0.492	1.67	2.24	0.746	2.13	0.0727	0.0596	0.384	0.266	0.0404	2.27	0.287	83.1	0.000590	0.501
6ZU1.25x048	6.000	1.250	0.048	0.1875	0.395	1.34	1.81	0.602	2.14	0.0590	0.0481	0.387	0.216	0.0328	1.83	0.288	83.1	0.000303	0.408
4ZU1.25x090	4.000	1.250	0.090	0.1875	0.551	1.87	1.21	0.603	1.48	0.105	0.0872	0.437	0.257	0.0481	1.26	0.296	77.5	0.00149	0.287
4ZU1.25x075	4.000	1.250	0.075	0.1875	0.462	1.57	1.02	0.510	1.49	0.0892	0.0735	0.439	0.217	0.0410	1.07	0.298	77.5	0.000866	0.245
4ZU1.25x060	4.000	1.250	0.060	0.1875	0.372	1.26	0.829	0.415	1.49	0.0727	0.0596	0.442	0.177	0.0334	0.868	0.300	77.5	0.000446	0.201
4ZU1.25x048	4.000	1.250	0.048	0.1875	0.299	1.02	0.671	0.336	1.50	0.0590	0.0481	0.444	0.143	0.0272	0.703	0.302	77.5	0.000229	0.164
4ZU1.25x036	4.000	1.250	0.036	0.1875	0.225	0.765	0.510	0.255	1.51	0.0449	0.0364	0.447	0.109	0.0207	0.534	0.303	77.5	0.0000972	0.126
3.625ZU1.25x090	3.625	1.250	0.090	0.1875	0.517	1.76	0.950	0.524	1.36	0.105	0.0872	0.451	0.232	0.0455	1.01	0.297	75.6	0.00140	0.228
3.625ZU1.25x075	3.625	1.250	0.075	0.1875	0.434	1.47	0.805	0.444	1.36	0.0892	0.0735	0.453	0.197	0.0388	0.856	0.299	75.6	0.000813	0.195
3.625ZU1.25x060	3.625	1.250	0.060	0.1875	0.349	1.19	0.655	0.361	1.37	0.0727	0.0596	0.456	0.160	0.0317	0.696	0.301	75.6	0.000419	0.161
3.625ZU1.25x048	3.625	1.250	0.048	0.1875	0.281	0.954	0.531	0.293	1.38	0.0590	0.0481	0.458	0.130	0.0258	0.564	0.303	75.6	0.000216	0.131
3.625ZU1.25x036	3.625	1.250	0.036	0.1875	0.212	0.719	0.403	0.222	1.38	0.0449	0.0364	0.461	0.0984	0.0197	0.428	0.305	75.6	0.0000914	0.100
2.5ZU1.25x090	2.500	1.250	0.090	0.1875	0.416	1.41	0.392	0.314	0.971	0.105	0.0872	0.503	0.158	0.0350	0.462	0.290	66.1	0.00112	0.0953
2.5ZU1.25x075	2.500	1.250	0.075	0.1875	0.349	1.19	0.334	0.267	0.978	0.0892	0.0735	0.505	0.134	0.0299	0.393	0.293	66.2	0.000655	0.0819
2.5ZU1.25x060	2.500	1.250	0.060	0.1875	0.282	0.957	0.273	0.218	0.984	0.0727	0.0596	0.508	0.109	0.0245	0.321	0.295	66.2	0.000338	0.0676
2.5ZU1.25x048	2.500	1.250	0.048	0.1875	0.227	0.771	0.222	0.178	0.990	0.0590	0.0481	0.510	0.0888	0.0200	0.261	0.297	66.3	0.000174	0.0554
2.5ZU1.25x036	2.500	1.250	0.036	0.1875	0.171	0.582	0.169	0.136	0.995	0.0449	0.0364	0.512	0.0675	0.0153	0.199	0.299	66.3	0.0000739	0.0426
1.5ZU1.25x090	1.500	1.250	0.090	0.1875	0.326	1.11	0.119	0.159	0.604	0.105	0.0872	0.568	0.0925	0.0193	0.205	0.243	47.2	0.000880	0.0275
1.5ZU1.25x075	1.500	1.250	0.075	0.1875	0.274	0.932	0.102	0.136	0.611	0.0892	0.0735	0.570	0.0788	0.0167	0.175	0.246	47.4	0.000514	0.0239
1.5ZU1.25x060	1.500	1.250	0.060	0.1875	0.222	0.753	0.0845	0.113	0.617	0.0727	0.0596	0.573	0.0645	0.0138	0.143	0.249	47.6	0.000266	0.0199
1.5ZU1.25x048	1.500	1.250	0.048	0.1875	0.179	0.608	0.0693	0.0924	0.623	0.0590	0.0481	0.575	0.0525	0.0113	0.117	0.252	47.8	0.000137	0.0164
1.5ZU1.25x036	1.500	1.250	0.036	0.1875	0.135	0.459	0.0532	0.0710	0.628	0.0449	0.0364	0.577	0.0401	0.00874	0.0894	0.254	48.0	0.0000583	0.0127

Table I - 6

Gross Section Properties

Equal Leg Angles With Lips





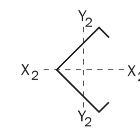
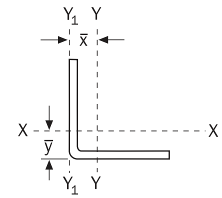
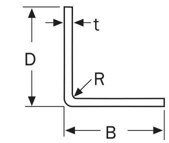
ID	Size		t	d	R	Area	wt/ft	Properties of Full Section												
								Axis x-x and y-y				r_{y1}	J	C_w	r_o				
	D	B						I_x	S_x	r	$\bar{x} = \bar{y}$					I_{y2}	r_{y2}	j	x_o
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.^{4}	in.	in.	in.
4LS4x135	4.000	4.000	0.135	0.500	0.1875	1.12	3.79	1.98	0.691	1.33	1.13	1.75	0.00678	0.0907	2.46	0.812	0.853	3.08	-1.58
4LS4x105	4.000	4.000	0.105	0.500	0.1875	0.879	2.99	1.59	0.556	1.35	1.13	1.76	0.00323	0.0791	2.49	0.657	0.864	3.10	-1.60
4LS4x090	4.000	4.000	0.090	0.500	0.1875	0.759	2.58	1.39	0.484	1.35	1.13	1.76	0.00205	0.0717	2.50	0.574	0.870	3.11	-1.61
4LS4x075	4.000	4.000	0.075	0.500	0.1875	0.636	2.16	1.18	0.410	1.36	1.13	1.77	0.00119	0.0631	2.52	0.488	0.875	3.12	-1.62
4LS4x060	4.000	4.000	0.060	0.500	0.1875	0.512	1.74	0.959	0.334	1.37	1.13	1.77	0.000615	0.0533	2.53	0.397	0.881	3.13	-1.63
3LS3x135	3.000	3.000	0.135	0.500	0.1875	0.846	2.88	0.850	0.402	1.00	0.886	1.34	0.00514	0.0479	1.89	0.359	0.652	2.34	-1.25
3LS3x105	3.000	3.000	0.105	0.500	0.1875	0.669	2.28	0.692	0.327	1.02	0.882	1.35	0.00246	0.0420	1.92	0.294	0.663	2.36	-1.27
3LS3x090	3.000	3.000	0.090	0.500	0.1875	0.579	1.97	0.607	0.286	1.02	0.881	1.35	0.00156	0.0382	1.93	0.258	0.668	2.37	-1.28
3LS3x075	3.000	3.000	0.075	0.500	0.1875	0.486	1.65	0.517	0.244	1.03	0.879	1.36	0.000912	0.0337	1.95	0.221	0.674	2.38	-1.29
3LS3x060	3.000	3.000	0.060	0.500	0.1875	0.392	1.33	0.423	0.199	1.04	0.878	1.36	0.000471	0.0286	1.96	0.181	0.679	2.39	-1.30
2.5LS2.5x135	2.500	2.500	0.135	0.500	0.1875	0.711	2.42	0.497	0.286	0.836	0.762	1.13	0.00432	0.0318	1.60	0.216	0.552	1.97	-1.08
2.5LS2.5x105	2.500	2.500	0.105	0.500	0.1875	0.564	1.92	0.408	0.234	0.850	0.759	1.14	0.00207	0.0281	1.63	0.179	0.562	1.98	-1.11
2.5LS2.5x090	2.500	2.500	0.090	0.500	0.1875	0.489	1.66	0.359	0.206	0.857	0.757	1.14	0.00132	0.0256	1.65	0.158	0.568	1.99	-1.12
2.5LS2.5x075	2.500	2.500	0.075	0.500	0.1875	0.411	1.40	0.307	0.176	0.864	0.755	1.15	0.000771	0.0226	1.66	0.135	0.573	2.00	-1.13
2.5LS2.5x060	2.500	2.500	0.060	0.500	0.1875	0.332	1.13	0.252	0.144	0.871	0.754	1.15	0.000399	0.0192	1.68	0.111	0.579	2.01	-1.14
2LS2x135	2.000	2.000	0.135	0.500	0.1875	0.576	1.96	0.257	0.189	0.668	0.639	0.924	0.00350	0.0192	1.32	0.118	0.452	1.58	-0.925
2LS2x105	2.000	2.000	0.105	0.500	0.1875	0.459	1.56	0.213	0.156	0.681	0.636	0.932	0.00169	0.0171	1.35	0.0984	0.463	1.60	-0.946
2LS2x090	2.000	2.000	0.090	0.500	0.1875	0.399	1.36	0.189	0.138	0.688	0.634	0.936	0.00108	0.0156	1.37	0.0874	0.468	1.61	-0.957
2LS2x075	2.000	2.000	0.075	0.500	0.1875	0.336	1.14	0.163	0.119	0.695	0.632	0.940	0.000631	0.0139	1.38	0.0754	0.473	1.62	-0.968
2LS2x060	2.000	2.000	0.060	0.500	0.1875	0.272	0.926	0.134	0.0980	0.702	0.631	0.943	0.000327	0.0118	1.39	0.0624	0.479	1.63	-0.979

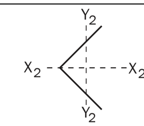
Table I - 7

Gross Section Properties

Equal Leg Angles Without Lips





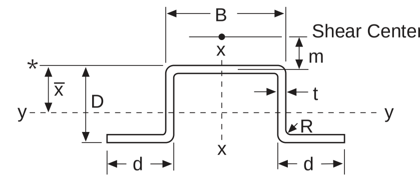


ID	Size		t	R	Area	wt/ft	Properties of Full Section: Axis x-x and Axis y-y				r_{y1}	J	C_w	r_o				
	D	B					I_x	S_x	r	$\bar{x} = \bar{y}$					I_{y2}	r_{y2}	j	x_o
	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.^{4}	in.	in.	in.
4LU4x135	4.000	4.000	0.135	0.1875	1.047	3.56	1.69	0.577	1.27	1.07	1.66	0.00636	0.000	2.29	0.653	0.790	2.87	-1.41
4LU4x105	4.000	4.000	0.105	0.1875	0.818	2.78	1.33	0.453	1.28	1.05	1.66	0.00301	0.000	2.29	0.515	0.794	2.87	-1.42
4LU4x090	4.000	4.000	0.090	0.1875	0.703	2.39	1.15	0.390	1.28	1.05	1.65	0.00190	0.000	2.30	0.445	0.796	2.88	-1.42
4LU4x075	4.000	4.000	0.075	0.1875	0.587	2.00	0.964	0.326	1.28	1.04	1.65	0.00110	0.000	2.30	0.374	0.798	2.88	-1.42
4LU4x060	4.000	4.000	0.060	0.1875	0.471	1.60	0.776	0.262	1.28	1.04	1.65	0.000565	0.000	2.31	0.301	0.800	2.88	-1.42
3LU3x135	3.000	3.000	0.135	0.1875	0.777	2.64	0.700	0.321	0.949	0.815	1.25	0.00472	0.000	1.71	0.266	0.585	2.16	-1.06
3LU3x105	3.000	3.000	0.105	0.1875	0.608	2.07	0.554	0.252	0.954	0.803	1.25	0.00224	0.000	1.72	0.211	0.589	2.17	-1.06
3LU3x090	3.000	3.000	0.090	0.1875	0.523	1.78	0.478	0.217	0.956	0.797	1.25	0.00141	0.000	1.72	0.183	0.591	2.17	-1.06
3LU3x075	3.000	3.000	0.075	0.1875	0.437	1.49	0.402	0.182	0.959	0.791	1.24	0.000820	0.000	1.73	0.154	0.593	2.17	-1.07
3LU3x060	3.000	3.000	0.060	0.1875	0.351	1.19	0.324	0.146	0.961	0.785	1.24	0.000421	0.000	1.73	0.124	0.595	2.18	-1.07
2.5LU2.5x135	2.500	2.500	0.135	0.1875	0.642	2.18	0.399	0.220	0.788	0.691	1.05	0.00390	0.000	1.42	0.149	0.482	1.81	-0.881
2.5LU2.5x105	2.500	2.500	0.105	0.1875	0.503	1.71	0.316	0.174	0.793	0.678	1.04	0.00185	0.000	1.43	0.119	0.486	1.82	-0.885
2.5LU2.5x090	2.500	2.500	0.090	0.1875	0.433	1.47	0.274	0.150	0.795	0.672	1.04	0.00117	0.000	1.43	0.103	0.488	1.82	-0.887
2.5LU2.5x075	2.500	2.500	0.075	0.1875	0.362	1.23	0.230	0.126	0.797	0.666	1.04	0.000679	0.000	1.44	0.0871	0.491	1.82	-0.889
2.5LU2.5x060	2.500	2.500	0.060	0.1875	0.291	0.989	0.186	0.101	0.800	0.660	1.04	0.000349	0.000	1.44	0.0705	0.493	1.82	-0.891
2LU2x135	2.000	2.000	0.135	0.1875	0.507	1.72	0.199	0.139	0.626	0.566	0.844	0.00308	0.000	1.13	0.0731	0.380	1.46	-0.705
2LU2x105	2.000	2.000	0.105	0.1875	0.398	1.35	0.159	0.110	0.631	0.554	0.840	0.00146	0.000	1.14	0.0586	0.384	1.47	-0.709
2LU2x090	2.000	2.000	0.090	0.1875	0.343	1.17	0.138	0.0947	0.633	0.548	0.837	0.000926	0.000	1.14	0.0511	0.386	1.47	-0.711
2LU2x075	2.000	2.000	0.075	0.1875	0.287	0.976	0.116	0.0796	0.636	0.541	0.835	0.000538	0.000	1.15	0.0432	0.388	1.47	-0.713
2LU2x060	2.000	2.000	0.060	0.1875	0.231	0.785	0.0940	0.0642	0.638	0.535	0.833	0.000277	0.000	1.15	0.0351	0.390	1.47	-0.715

Table I - 8

Gross Section Properties

Hat Sections Without Lips



* Note vertical orientation of x-axis.

ID	Dimensions							Properties of Full Section												
								Axis x-x			Axis y-y									
	D	B	t	d	R	Area	wt/ft	I_x	S_x	r_x	I_y	S_y	r_y	$\bar{x}$	m	J	C_w	j	r_o	x_o
	in.	in.	in.	in.	in.	in.^{2}	lb	in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.	in.	in.^{4}	in.^{6}	in.	in.	in.
10HU5x075	10.000	5.000	0.075	1.050	0.1875	1.98	6.74	11.0	3.15	2.35	23.7	4.28	3.46	4.46	4.80	0.00371	98.4	9.70	10.1	-9.22
8HU12x135	8.000	12.000	0.135	1.670	0.1875	4.10	13.9	111	14.8	5.21	36.3	7.09	2.98	2.88	3.63	0.0249	762	9.06	8.80	-6.44
8HU12x105	8.000	12.000	0.105	1.340	0.1875	3.13	10.7	83.6	11.6	5.17	26.8	5.13	2.93	2.77	3.59	0.0115	597	8.93	8.66	-6.31
8HU8x105	8.000	8.000	0.105	1.340	0.1875	2.71	9.23	35.2	6.73	3.60	23.3	4.83	2.93	3.19	3.74	0.00997	222	8.17	8.29	-6.87
8HU8x075	8.000	8.000	0.075	0.980	0.1875	1.90	6.44	24.1	4.92	3.57	15.6	3.16	2.87	3.05	3.71	0.00355	161	8.05	8.14	-6.73
8HU4x075	8.000	4.000	0.075	0.980	0.1875	1.60	5.42	5.69	1.96	1.89	12.4	2.82	2.78	3.62	3.81	0.00299	31.9	7.77	8.12	-7.39
8HU4x060	8.000	4.000	0.060	0.840	0.1875	1.26	4.30	4.47	1.61	1.88	9.66	2.18	2.77	3.56	3.84	0.00152	25.8	7.76	8.10	-7.37
6HU9x135	6.000	9.000	0.135	1.670	0.1875	3.15	10.7	49.7	8.24	3.97	16.4	4.43	2.28	2.29	2.73	0.0192	177	6.97	6.75	-4.95
6HU9x105	6.000	9.000	0.105	1.340	0.1875	2.40	8.16	36.8	6.42	3.92	12.1	3.16	2.24	2.19	2.73	0.00882	140	6.85	6.63	-4.86
6HU6x105	6.000	6.000	0.105	1.340	0.1875	2.08	7.09	15.7	3.71	2.74	10.4	2.98	2.24	2.51	2.78	0.00766	51.6	6.21	6.32	-5.23
6HU6x075	6.000	6.000	0.075	0.915	0.1875	1.44	4.88	10.4	2.70	2.69	6.80	1.87	2.18	2.36	2.80	0.00269	37.7	6.11	6.18	-5.12
6HU3x075	6.000	3.000	0.075	0.915	0.1875	1.21	4.12	2.48	1.06	1.43	5.36	1.67	2.11	2.79	2.79	0.00227	7.55	5.81	6.10	-5.54
6HU3x060	6.000	3.000	0.060	0.760	0.1875	0.954	3.24	1.92	0.872	1.42	4.15	1.27	2.09	2.72	2.85	0.00115	6.04	5.83	6.09	-5.55
6HU3x048	6.000	3.000	0.048	0.660	0.1875	0.757	2.57	1.51	0.714	1.41	3.25	0.979	2.07	2.68	2.88	0.000581	4.88	5.83	6.08	-5.54
4HU6x135	4.000	6.000	0.135	1.670	0.1875	2.21	7.51	16.9	3.72	2.76	5.42	2.34	1.57	1.69	1.74	0.0134	22.6	4.81	4.63	-3.36
4HU6x105	4.000	6.000	0.105	1.340	0.1875	1.66	5.66	12.0	2.85	2.69	3.96	1.64	1.54	1.59	1.80	0.00612	17.9	4.74	4.56	-3.34
4HU4x105	4.000	4.000	0.105	1.340	0.1875	1.45	4.94	5.31	1.64	1.91	3.39	1.55	1.53	1.81	1.73	0.00534	6.89	4.17	4.26	-3.49
4HU4x075	4.000	4.000	0.075	0.915	0.1875	0.986	3.35	3.30	1.16	1.83	2.18	0.937	1.49	1.68	1.85	0.00185	4.83	4.15	4.21	-3.49
4HU2x075	4.000	2.000	0.075	0.915	0.1875	0.836	2.84	0.830	0.451	0.997	1.70	0.839	1.43	1.97	1.65	0.00157	1.15	3.74	3.99	-3.59
4HU2x060	4.000	2.000	0.060	0.750	0.1875	0.653	2.22	0.617	0.365	0.972	1.30	0.623	1.41	1.91	1.78	0.000784	0.829	3.83	4.05	-3.66
4HU2x048	4.000	2.000	0.048	0.618	0.1875	0.513	1.74	0.469	0.299	0.956	1.00	0.468	1.40	1.86	1.86	0.000394	0.640	3.88	4.06	-3.69
3HU4.5x135	3.000	4.500	0.135	1.670	0.1875	1.74	5.90	8.28	2.19	2.18	2.47	1.52	1.19	1.37	1.18	0.0105	5.66	3.69	3.52	-2.49
3HU4.5x105	3.000	4.500	0.105	1.340	0.1875	1.30	4.41	5.69	1.63	2.10	1.80	1.05	1.18	1.29	1.29	0.00477	4.20	3.64	3.48	-2.52
3HU3x105	3.000	3.000	0.105	1.340	0.1875	1.14	3.87	2.62	0.956	1.52	1.53	0.992	1.16	1.46	1.13	0.00419	1.90	3.08	3.17	-2.53
3HU3x075	3.000	3.000	0.075	0.915	0.1875	0.761	2.59	1.52	0.648	1.41	0.977	0.585	1.13	1.33	1.33	0.00143	1.14	3.13	3.19	-2.62

2.4 Steel Deck

Steel decks are at times used for architectural application; however, they are fundamentally structural products. The structural capabilities (strength and stiffness) are determined using the *Specification*, which also provides base steel specifications. Design may use either Allowable Strength Design (ASD) or Load and Resistance Factor Design (LRFD) techniques. The usual deck products are roof deck, form deck and composite floor deck. In addition to supporting gravity and wind uplift loads, steel deck profiles can also serve to resist in-plane loading, i.e. diaphragm loads. The most common types of steel deck are discussed in Section 2.4.1.

One of the important items to consider for all deck products is the required service life and the environment in which the deck product must exist. For instance, if a roof deck is to have insulation board applied with fasteners that penetrate the deck, then consideration should be given to corrosion of the fasteners and the deck. The same reasoning applies to form deck and floor deck in humid areas or areas subject to water application. Insurance requirements and fire ratings can also affect the finish selection.

2.4.1 Deck Profiles

Figure 2.4-1 shows cross sections of industry standard steel deck profiles. These profiles are merely representative of what is available. Consult the literature of manufacturers to obtain further information about these and other available deck profiles.

The different profiles lend themselves to different uses:

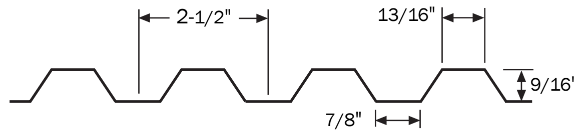
(a) Form Deck: Form deck is commonly used to span between floor joists to serve as form work for cast in place concrete floor systems. The 9/16 inch deep profile shown is the shallowest available. Deeper profiles are available for longer spans and heavier loads. Flutes may be trapezoidal as shown or sinusoidal.

(b) Narrow Rib Deck (NR): Narrow rib deck is used as roof decking in climatic areas where minimal roof insulation is required. The narrow rib provides a small span that thin rigid insulation can bridge. Narrow rib deck is the least efficient structurally but has the advantage of easy attachment of roofing materials since a high percentage of the material is in the roof plane.

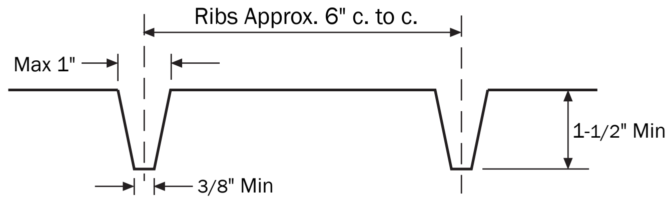
(c) Intermediate Rib Deck (IR): Intermediate rib deck is used as roof decking. It is somewhat more efficient structurally than narrow rib deck, but requires thicker insulation to span the wider flute.

(d) Wide Rib Deck (WR): Wide rib deck is used as roof deck where insulation can span the wide flutes. Because of energy demands, this is the most common profile. It is the most structurally efficient cross section of the 1.5 inch roof deck profiles.

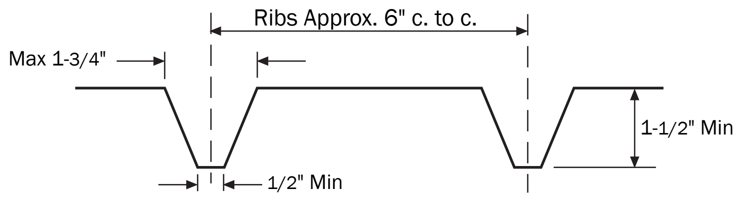
(e) Deep Rib Deck (3DR): Deep rib deck is used where long spans between joists or purlins occur and/or the deck spans are subject to larger loads.



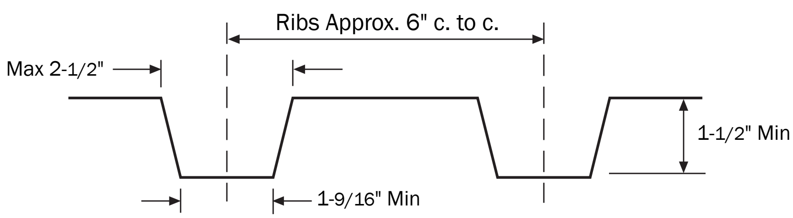
(a) 9/16" Form Deck (Representative)



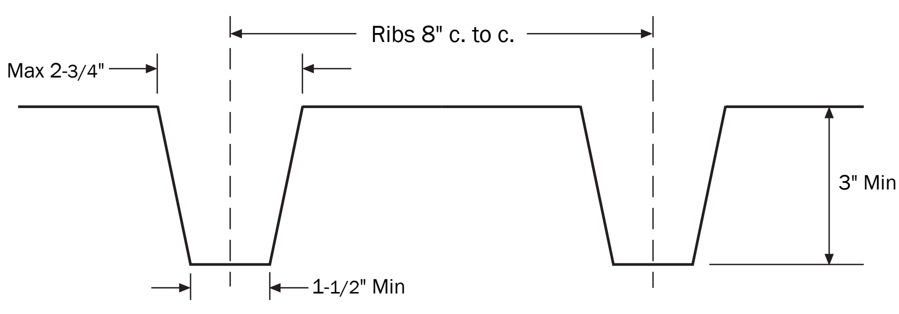
(b) Narrow Rib Deck Type NR



(c) Intermediate Rib Deck Type IR



(d) Wide Rib Deck Type WR



(e) Deep Rib Deck Type 3DR

Figure 2.4-1 Deck Profiles

In addition to these industry standard profiles, manufacturers produce proprietary steel deck profiles that provide special functional enhancements, such as acoustical absorption capability and cellular raceway features.

The thickness of steel deck is commonly designated by gage. The equivalent design thicknesses given in Section 2.4.3 are the minimum allowed at each designated gage by the Steel Deck Institute (SDI) in the *ANSI/SDI-RD1.0 Standard for Steel Roof Deck*. Actual material thicknesses provided by manufacturers will vary. Steel must conform to *Specification* Section A2.

2.4.2 Maximum Spans

Recommended Maximum Spans for Construction and Maintenance Loads

Type*		Span Condition	Maximum Recommended Spans	
			Span ft-in.	Roof Deck Cantilever ft-in.
Narrow Rib Deck	NR22	1	3′-10″	
	NR22	2 or more	4′-9″	1′-0″
	NR20	1	4′-10″	
	NR20	2 or more	5′-11″	1′-2″
	NR18	1	5′-11″	
	NR18	2 or more	6′-11″	1′-7″
Intermediate Rib Deck	IR22	1	4′-6″	
	IR22	2 or more	5′-6″	1′-2″
	IR20	1	5′-3″	
	IR20	2 or more	6′-3″	1′-5″
	IR18	1	6′-2″	
	IR18	2 or more	7′-4″	1′-10″
Wide Rib Deck	WR22	1	5′-6″	
	WR22	2 or more	6′-6″	1′-11″
	WR20	1	6′-3″	
	WR20	2 or more	7′-5″	2′-4″
	WR18	1	7′-6″	
	WR18	2 or more	8′-10″	2′-10″
Deep Rib Deck	3DR22	1	11′-0″	
	3DR22	2 or more	13′-0″	3′-5″
	3DR20	1	12′-6″	
	3DR20	2 or more	14′-8″	3′-11″
	3DR18	1	15′-0″	
	3DR18	2 or more	17′-8″	4′-9″

* Deck section properties are provided in Section 2.4.3

Construction and maintenance loads

Spans are governed by a maximum stress of $0.7F_y$ and a maximum deflection of L/240 with a 200 pound concentrated load at midspan on a 12 inch wide section of deck. The table above is based on F_y = 33 ksi.

If the designer contemplates loads of greater magnitude, spans shall be decreased or the thickness of the steel deck increased as required.

All loads shall be distributed by appropriate means to prevent damage to the completed assembly during construction.

Do not walk or stand on deck until it is fastened in accordance with the Steel Deck Institute Design Manual, Publication No. 31.

Cantilever loads

Construction phase load of 10 psf on adjacent span and cantilever plus 200 pound load at end of cantilever with a stress limit of $0.7F_y$ for ASD. The table is based on F_y = 33 ksi.

Service load of 45 psf on adjacent span and cantilever plus 100 pound load at end of cantilever with a stress limit of $0.6F_y$.

Deflection limited to 1/240 of adjacent span for interior span and deflection at end of cantilever to 1/120 of overhang.

Notes:

1. Adjacent span: Limited to those spans shown in Section 2.4 of the SDI Roof Deck Standard. In those instances where the adjacent span is less than 3 times the cantilever span, the individual manufacturer should be consulted for the appropriate cantilever span.
2. Sidelaps must be attached at the end of the cantilever and at a maximum of 12 inches on center from the end.
3. No permanent suspended loads are to be supported by the steel deck.
4. The deck must be completely attached to the supports and at the sidelaps before any load is applied to the cantilever.

2.4.3 Section Properties

The Steel Deck Institute (SDI) used the most conservative combinations of the dimensions for each roof deck profile shown in Figure 2.4-1 (with the exception of the form deck) to calculate the section properties listed in the table below. The values are therefore not representative of any one manufacturer but represent the lowest value that might occur. As a result, load tables based on these properties are conservative. Form deck profiles vary greatly and their profiles are not established by SDI. The form deck profile shown is an actual manufacturer's product. These properties can be considered as representative, but actual properties may be higher or lower.

In the tables below, the I and S_t values are for compression on the top. The S_b values are for compression on the bottom. The weight provided is for dead load calculations; it should not be used as a basis for ordering. The values given are based on steel with yield stress of 33 ksi, with the exception of the form deck values, which are based on 80 ksi.

Steel Deck - Section Properties

Type	Common Designation	Design Thickness in.	Weight lb/ft^2		I in.^{4}/ft	S_t in.^{3}/ft	S_b in.^{3}/ft
			Painted	Galvanized			
9/16″ Form Deck	28 gage	0.0149	0.8	0.9	0.012	0.036	0.037
	26 gage	0.0179	0.9	1.0	0.015	0.046	0.047
	24 gage	0.0239	1.2	1.3	0.020	0.065	0.064
	22 gage	0.0295	1.5	1.6	0.025	0.080	0.079
1-1/2″ Narrow Rib Deck	NR22	0.0295	1.6	1.7	0.099	0.089	0.098
	NR20	0.0358	2.0	2.1	0.128	0.111	0.118
	NR18	0.0474	2.6	2.7	0.181	0.152	0.157
1-1/2″ Intermediate Rib Deck	IR22	0.0295	1.6	1.7	0.108	0.102	0.110
	IR20	0.0358	2.0	2.1	0.139	0.127	0.134
	IR18	0.0474	2.6	2.7	0.196	0.173	0.177
1-1/2″ Wide Rib Deck	WR22	0.0295	1.7	1.8	0.152	0.182	0.184
	WR20	0.0358	2.1	2.2	0.198	0.226	0.237
	WR18	0.0474	2.8	2.9	0.284	0.307	0.316
3″ Deep Rib Deck	3DR22	0.0295	2.1	2.2	0.551	0.321	0.369
	3DR20	0.0358	2.5	2.6	0.714	0.400	0.449
	3DR18	0.0474	3.3	3.4	1.036	0.550	0.594

SECTION 3 - CALCULATION OF SECTION PROPERTIES

3.1 Linear Method For Computing Properties Of Formed Sections

Computation of properties of formed sections may be simplified by using a so-called linear method, in which the material of the section is considered concentrated along the centerline of the steel sheet and the area elements replaced by straight or curved "line elements." The thickness dimension, t, is introduced after the linear computations have been completed.

The total area of the section is found from the relation: $\text{Area} = \text{Lt}$, where L is the total length of all line elements.

The moment of inertia of the section, I, is found from the relation: $I = I't$, where I' is the moment of inertia of the centerline of the steel sheet. The section modulus is computed as usual by dividing I or $I't$ by the distance from the neutral axis to *the extreme fiber*, not to the centerline of the extreme element.

First power dimensions, such as x, y, and r (radius of gyration) are obtained directly by the linear method and do not involve the thickness dimension.

When the flat width, w, of a stiffened compression element is reduced for design purposes, the effective design width, b, is used directly to compute the total effective length L_{eff} of the line elements.

The elements into which most sections may be divided for application of the linear method consist of straight lines and circular arcs. For convenient reference, the moments of inertia and location of centroid of such elements are identified in the sketches and equations in Section 3.2.

The equations for line elements are exact, since the line as such has no thickness dimension; but in computing the properties of an actual section, where the line element represents an actual element with a thickness dimension, the results will be approximate for the following reasons:

(1) The moment of inertia of a straight actual element about its longitudinal axis is considered negligible.

(2) The moment of inertia of a straight actual element inclined to the axes of reference is slightly larger than that of the corresponding line element, but for elements of like length the error involved is even less than the error involved in neglecting the moment of inertia of the element about its longitudinal axis. Obviously, the error disappears when the element is normal to the axis.

(3) Small errors are involved in using the properties of a linear arc to find those of an actual corner, but with the usual small corner radii the error in the location of the centroid of the corner is of little importance, and the moment of inertia generally negligible. When the mean radius of a circular element is over four times its thickness, as for tubular sections and for sheets with circular corrugations, the errors in using linear arc properties practically disappear.

Using the computed values of I_x, I_y, and I_{xy} the moment of inertia about principal axes of the section can be calculated by the following equation:

$$I_{\substack{Max,\\ Min}} = \frac{I_x + I_y}{2} \pm \sqrt{\left(\frac{I_x - I_y}{2}\right)^2 + I_{xy}^{\;2}}$$

where I_x and I_y are the moment of inertia of the section about x- and y-axis, respectively and I_{xy} is the product of inertia.

The angle between the x-axis and the minor principal axis is

$$\theta = \frac{\pi}{2} + \frac{1}{2}\tan^{-1}\left[\frac{2I_{xy}}{I_y - I_x}\right] \quad \text{(in radians)}$$

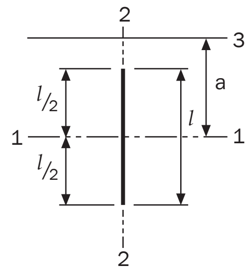
3.2 Properties of Line Elements

3.2.1 Straight Line Elements

Moments of inertia of straight line elements can be calculated using the equations given below:

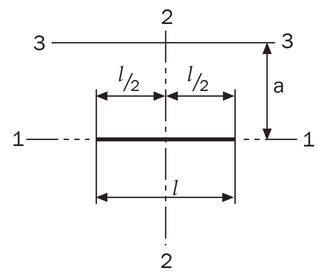
$$I_1 = \frac{l^3}{12} \qquad I_2 = 0$$

$$I_3 = la^2 + \frac{l^3}{12} = l\left(a^2 + \frac{l^2}{12}\right)$$



$$I_1 = 0 \qquad I_2 = \frac{l^3}{12}$$

$$I_3 = la^2$$

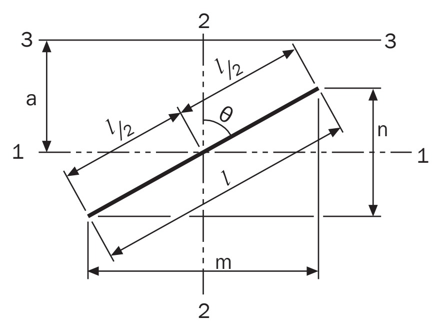


$$I_1 = \left[\frac{\cos^2\theta}{12}\right] l^3 = \frac{ln^2}{12}$$

$$I_2 = \left[\frac{\sin^2\theta}{12}\right] l^3 = \frac{lm^2}{12}$$

$$I_{12} = \left[\frac{\sin\theta\cos\theta}{12}\right] l^3 = \frac{lmn}{12}$$

$$I_3 = la^2 + \frac{ln^2}{12} = l\left(a^2 + \frac{n^2}{12}\right)$$



3.2.2 Circular Line Elements

Moments of inertia of circular line elements can be calculated using the equations given below:

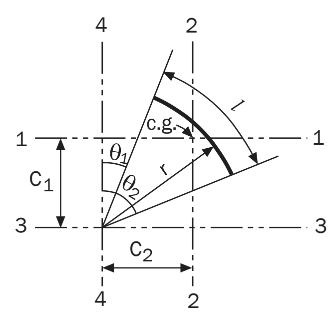
R = inside radius

r = median radius

θ (expressed in radians) = $\dfrac{\pi\theta}{180}$ (expressed in degrees)

$l = (\theta_2 - \theta_1)r$

$c_1 = \dfrac{\sin\theta_2 - \sin\theta_1}{\theta_2 - \theta_1} r \qquad c_2 = \dfrac{\cos\theta_1 - \cos\theta_2}{\theta_2 - \theta_1} r$



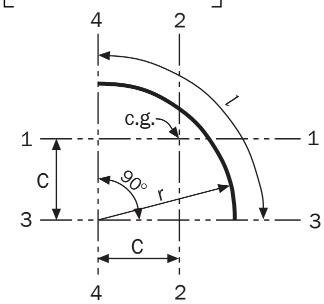
$$I_1 = \left[\frac{\theta_2 - \theta_1 + \sin\theta_2\cos\theta_2 - \sin\theta_1\cos\theta_1}{2} - \frac{(\sin\theta_2 - \sin\theta_1)^2}{\theta_2 - \theta_1}\right] r^3$$

$$I_2 = \left[\frac{\theta_2-\theta_1-\sin\theta_2\cos\theta_2+\sin\theta_1\cos\theta_1}{2}-\frac{(\cos\theta_1-\cos\theta_2)^2}{\theta_2-\theta_1}\right]r^3$$

$$I_{12} = \left[\frac{(\sin^2\theta_2-\sin^2\theta_1)^2}{2}+\frac{(\sin\theta_2-\sin\theta_1)(\cos\theta_2-\cos\theta_1)}{\theta_2-\theta_1}\right]r^3$$

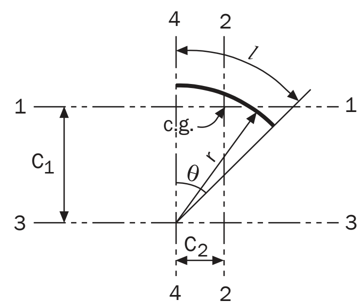
$$I_3 = \left[\frac{\theta_2-\theta_1+\sin\theta_2\cos\theta_2-\sin\theta_1\cos\theta_1}{2}\right]r^3$$

$$I_4 = \left[\frac{\theta_2-\theta_1-\sin\theta_2\cos\theta_2+\sin\theta_1\cos\theta_1}{2}\right]r^3 \qquad I_{34} = \left[\frac{\sin^2\theta_2-\sin^2\theta_1}{2}\right]r^3$$



Case I: $\theta_1 = 0;\ \theta = 90°$

$$l = \pi r/2 = 1.57r$$
$$c = 0.637r$$
$$I_1 = I_2 = 0.149r^3$$
$$I_{12} = -0.137r^3$$
$$I_3 = I_4 = 0.785r^3$$
$$I_{34} = 0.5r^3$$



Case II: $\theta_1 = 0;\ \theta = \theta$

$$l = \theta r$$
$$c_1 = \frac{r\sin\theta}{\theta}$$
$$c_1 = \frac{r(1\text{-}\cos\theta)}{\theta}$$

$$I_1 = \left[\frac{\theta+\sin\theta\cos\theta}{2}-\frac{\sin^2\theta}{\theta}\right]r^3 \qquad I_2 = \left[\frac{\theta-\sin\theta\cos\theta}{2}-\frac{(1-\cos\theta)^2}{\theta}\right]r^3$$

$$I_{12} = \left[\frac{\sin^2\theta}{2}+\frac{\sin\theta(\cos\theta-1)}{\theta}\right]r^3$$

$$I_3 = \left[\frac{\theta+\sin\theta\cos\theta}{2}\right]r^3 \qquad I_4 = \left[\frac{\theta-\sin\theta\cos\theta}{2}\right]r^3$$

$$I_{34} = \left[\frac{\sin^2\theta}{2}\right]r^3$$

3.3 Properties of Sections

Section properties of some sections can be calculated using the equations given below. The following are to be noted:

(1) Three different types of dimensions are used: capital letters (A) for outside dimensions, lower case barred letters ($\bar{a}$) for centerline dimensions, lower case letters (a) for flat dimensions. The flat dimensions are required to obtain properties such as moment of inertia, I, where corners are assumed to be round. The centerline dimensions are needed for torsional properties such as C_w where corners are assumed to be square. The outside dimensions are shown because they are the dimensions usually given in tables.

(2) All expressions consider the sections to contain round corners with the exception of those for some torsional properties (m, j and C_w). These expressions are based on a square corner approximation with the exception that round corner values are used for quantities such as area and moment of inertia which appear in the torsional property expressions. However, nominal stresses calculated by this procedure are sufficiently accurate for routine engineering design of sections with small ratios of corner radius to thickness.

(3) In the moment of inertia calculations, all quantities are accounted for except the moment of inertia of a flat element about its own axis when this is the weak axis. Moments of inertia of corners about their own axis are included to provide for the case of sections with large corner radii.

(4) All expressions are given for the full, unreduced sections.

3.3.1 Equal Leg Angles (Singly-Symmetric) With and Without Lips

1. Basic parameters

$$a = A' - \left[r + t/2 + \alpha\left(r + t/2\right)\right]^*$$

$$\bar{a} = A' - \left[t/2 + \alpha t/2\right]$$

$$c = \alpha\left[C' - \left(r + t/2\right)\right]$$

$$\bar{c} = \alpha\left[C' - t/2\right]$$

$$u = \pi r/2 = 1.57r$$

2. Cross-Sectional area

$$A = t\left[2a + u + \alpha\left(2c + 2u\right)\right]$$

3. Distance between centroid and centerlines of webs

$$\bar{x}_c = \bar{y}_c = \frac{t}{A}\left\{a\left(\frac{a}{2} + r\right) + u\left(0.363r\right) + \alpha\left[c\left(a + \frac{c}{2} + 3r\right) + u\left(a + 2r\right)\right]\right\}$$

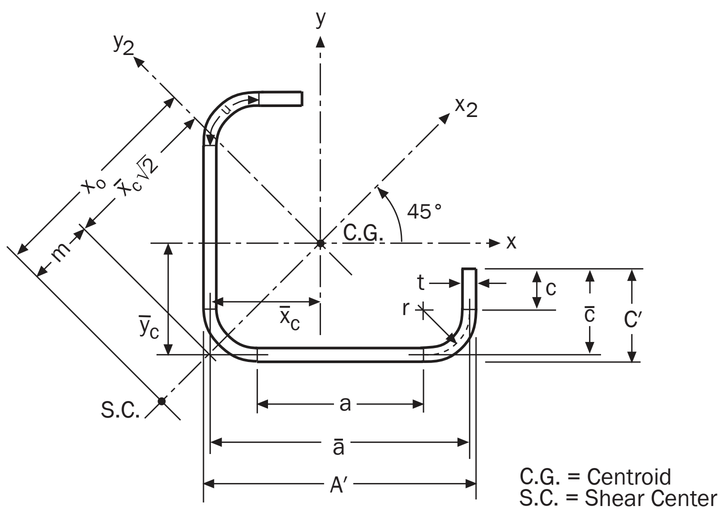


Figure 3.3.1-1
Equal Leg Angle (Singly Symmetric) With Lips

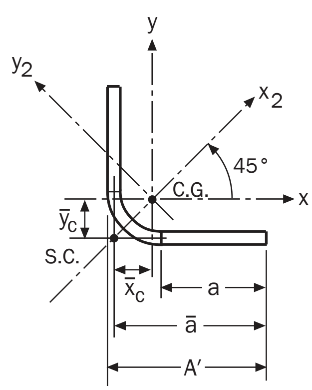


Figure 3.3.1-2
Equal Leg Angle (Singly Symmetric) Without Lips

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* For sections with lips, $\alpha = 1.0$; for sections without lips, $\alpha = 0$.

4. Distance between centroid and outside of webs

$$\bar{x} = \bar{y} = \bar{x}_c + \frac{t}{2}$$

5. Moment of inertia about x and y axes

$$I_x = I_y = t\left\{\begin{array}{l} a\left(\frac{a}{2}+r\right)^2 + \frac{a^3}{12} + u(0.363r)^2 + 0.149r^3 \\ +\alpha\left[\begin{array}{l} c\left(a+2r\right)^2 + \frac{c^3}{12} + c\left(\frac{c}{2}+r\right)^2 + u\left(a+1.637r\right)^2 \\ +u\left(0.363r\right)^2 + \left(2\right)\left(0.149\right)r^3 \end{array}\right] \end{array}\right\} - A\bar{x}_c^2$$

6. Product of inertia about x and y axes

$$I_{xy} = t\left\{\begin{array}{l} -0.137r^3 + u\left(0.363r\right)^2 \\ +2\alpha\left[c\left(a+2r\right)\left(\frac{c}{2}+r\right) + 0.137r^3 + u\left(a+1.637r\right)\left(0.363r\right)\right] \end{array}\right\} - A\bar{x}_c\bar{y}_c$$

7. Moment of inertia about y_2-axis

$$I_{y2} = I_x + I_{xy}$$

8. Distance between shear center and centerline of square corner

$$m = \frac{\bar{a}\bar{c}^2\sqrt{2}}{2}\frac{\left(3\bar{a}-2\bar{c}\right)}{\left[2\bar{a}^3-\left(\bar{a}-\bar{c}\right)^3\right]}$$

9. St. Venant torsion constant

$$J = \frac{t^3}{3}\left[2a+u+\alpha\left(2c+2u\right)\right]$$

10. Warping constant

$$C_w = \frac{\bar{a}^4\bar{c}^3 t}{6}\frac{\left(4\bar{a}+3\bar{c}\right)}{\left[2\bar{a}^3-\left(\bar{a}-\bar{c}\right)^3\right]}$$

11. Distance from centroid to shear center*

$$x_o = -\left(\bar{x}_c\sqrt{2}+m\right)$$

12. Parameter used to determine elastic critical moment

$$j = \frac{\sqrt{2}t}{48I_{y2}}\left(\bar{a}^4 + 4\bar{a}^3\bar{c} - 6\bar{a}^2\bar{c}^2 + \bar{c}^4\right) - x_o$$

* Negative sign indicates x_o is measured in negative x_2 direction.

3.3.2 C-Sections (Singly-Symmetric) With and Without Lips and Hat Sections (Singly-Symmetric)

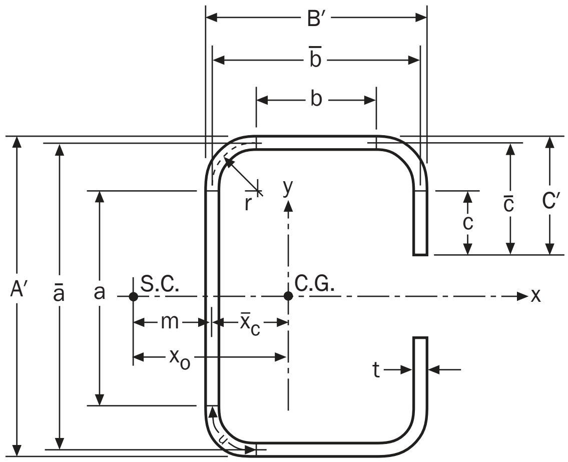


Figure 3.3.2-1
C-Section (Singly Symmetric) With Lips

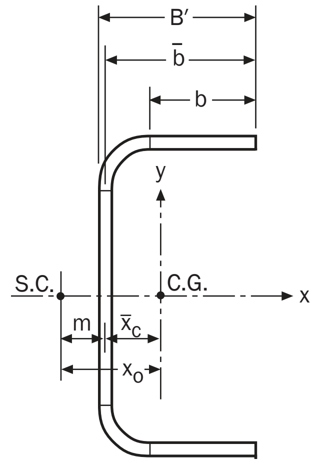
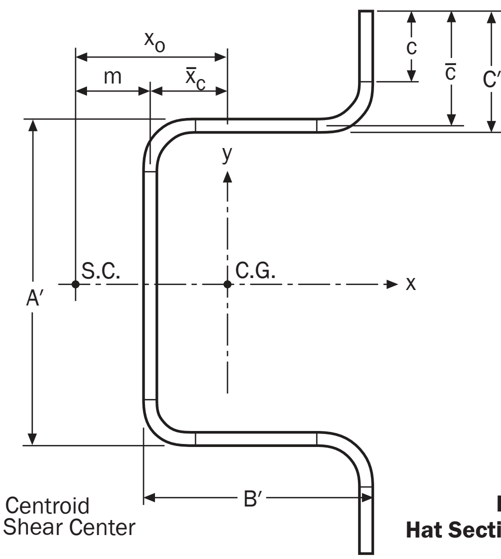


Figure 3.3.2-2
C-Section (Singly Symmetric) Without Lips



C.G. = Centroid
S.C. = Shear Center

Figure 3.3.2-3
Hat Section (Singly Symmetric)

1. Basic Parameters

$a = A' - (2r + t)$

$\bar{a} = A' - t$

$b = B' - \left[r + t/2 + \alpha(r + t/2)\right]$ *

$\bar{b} = B' - (t/2 + \alpha t/2)$

$c = \alpha\left[C' - (r + t/2)\right]$

$\bar{c} = \alpha(C' - t/2)$

$u = \pi r/2 = 1.57r$

* For sections with lips, α = 1.0; for sections without lips, α = 0.

2. Cross-sectional area

$$A = t\left[a + 2b + 2u + \alpha\left(2c + 2u\right)\right]$$

3. Moment of inertia about x-axis

Channel:

$$I_x = 2t\left\{\begin{array}{l} 0.0417a^3 + b\left(a/2 + r\right)^2 + u\left(a/2 + 0.637r\right)^2 + 0.149r^3 \\ +\alpha\left[0.0833c^3 + \dfrac{c}{4}\left(a - c\right)^2 + u\left(\dfrac{a}{2} + 0.637r\right)^2 + 0.149r^3\right] \end{array}\right\}$$

Hat Section:

$$I_x = 2t\left\{\begin{array}{l} 0.0417a^3 + b\left(a/2 + r\right)^2 + u\left(a/2 + 0.637r\right)^2 + 0.149r^3 \\ +\alpha\left[0.0833c^3 + \dfrac{c}{4}\left(a + c + 4r\right)^2 + u\left(\dfrac{a}{2} + 1.363r\right)^2 + 0.149r^3\right] \end{array}\right\}$$

4. Distance between centroid and web centerline

$$\bar{x}_c = \frac{2t}{A}\left\{b\left(b/2 + r\right) + u\left(0.363r\right) + \alpha\left[u\left(b + 1.637r\right) + c\left(b + 2r\right)\right]\right\}$$

5. Distance between centroid and outside of web

$$\bar{x} = \bar{x}_c + \frac{t}{2}$$

6. Moment of inertia about y-axis

$$I_y = 2t\left\{\begin{array}{l} b\left(b/2 + r\right)^2 + b^3/12 + 0.356r^3 \\ +\alpha\left[c\left(b + 2r\right)^2 + u\left(b + 1.637r\right)^2 + 0.149r^3\right] \end{array}\right\} - A\bar{x}_c^{\,2}$$

7. Distance between shear center and web centerline

a) Channel:

$$m = \bar{b}\left[\frac{3\bar{a}^2\bar{b} + \alpha\bar{c}\left(6\bar{a}^2 - 8\bar{c}^2\right)}{\bar{a}^3 + 6\bar{a}^2\bar{b} + \alpha\bar{c}\left(8\bar{c}^2 - 12\bar{a}\bar{c} + 6\bar{a}^2\right)}\right]$$

b) Hat Section:

$$m = \bar{b}\left[\frac{3\bar{a}^2\bar{b} + \alpha\bar{c}\left(6\bar{a}^2 - 8\bar{c}^2\right)}{\bar{a}^3 + 6\bar{a}^2\bar{b} + \alpha\bar{c}\left(8\bar{c}^2 + 12\bar{a}\bar{c} + 6\bar{a}^2\right)}\right]$$

8. Distance between centroid and shear center

$$x_o = -\left(\bar{x}_c + m\right)^*$$

9. St. Venant torsion constant

$$J = \frac{t^3}{3}\left[a + 2b + 2u + \alpha\left(2c + 2u\right)\right]$$

* Negative sign indicates x_o is measured in negative x direction.

10. Warping constant

a) Channel:

$$C_w = \frac{\bar{a}^2\bar{b}^2 t}{12}\left\{\frac{2\bar{a}^3\bar{b} + 3\bar{a}^2\bar{b}^2 + \alpha\begin{bmatrix}48\bar{c}^4 + 112\bar{b}\bar{c}^3 + 8\bar{a}\bar{c}^3 + 48\bar{a}\bar{b}\bar{c}^2 \\ +12\bar{a}^2\bar{c}^2 + 12\bar{a}^2\bar{b}\bar{c} + 6\bar{a}^3\bar{c}\end{bmatrix}}{6\bar{a}^2\bar{b} + \left(\bar{a} + \alpha 2\bar{c}\right)^3 - \alpha 24\bar{a}\bar{c}^2}\right\}$$

b) Hat section:

$$C_w = \frac{\bar{a}^2\bar{b}^2 t}{12}\left\{\frac{2\bar{a}^3\bar{b} + 3\bar{a}^2\bar{b}^2 + \alpha\begin{bmatrix}48\bar{c}^4 + 112\bar{b}\bar{c}^3 + 8\bar{a}\bar{c}^3 - 48\bar{a}\bar{b}\bar{c}^2 \\ -12\bar{a}^2\bar{c}^2 + 12\bar{a}^2\bar{b}\bar{c} + 6\bar{a}^3\bar{c}\end{bmatrix}}{6\bar{a}^2\bar{b} + \left(\bar{a} + \alpha 2\bar{c}\right)^3}\right\}$$

11. Parameter β_w

$$\beta_w = -\left[\frac{t\bar{x}_c\bar{a}^3}{12} + t\bar{x}_c^{\,3}\bar{a}\right]$$

12. Parameter β_f

$$\beta_f = \frac{t}{2}\left[\left(\bar{b} - \bar{x}_c\right)^4 - \bar{x}_c^{\,4}\right] + \frac{t\bar{a}^2}{4}\left[\left(\bar{b} - \bar{x}_c\right)^2 - \bar{x}_c^{\,2}\right]$$

13. Parameter β_l

a) Channel: $\beta_l = \alpha\left\{2\bar{c}t\left(\bar{b} - \bar{x}_c\right)^3 + \frac{2}{3}t\left(\bar{b} - \bar{x}_c\right)\left[\left(\bar{a}/2\right)^3 - \left(\bar{a}/2 - \bar{c}\right)^3\right]\right\}$

b) Hat section: $\beta_l = 2\bar{c}t\left(\bar{b} - \bar{x}_c\right)^3 + \frac{2}{3}t\left(\bar{b} - \bar{x}_c\right)\left[\left(\bar{a}/2 + \bar{c}\right)^3 - \left(\bar{a}/2\right)^3\right]$

14. Parameter used in determination of elastic critical moment

$$j = \frac{1}{2I_y}\left(\beta_w + \beta_f + \beta_l\right) - x_o$$

3.3.3 I-Sections With Unequal Flanges (Singly-Symmetric) and T-Sections (Singly-Symmetric)

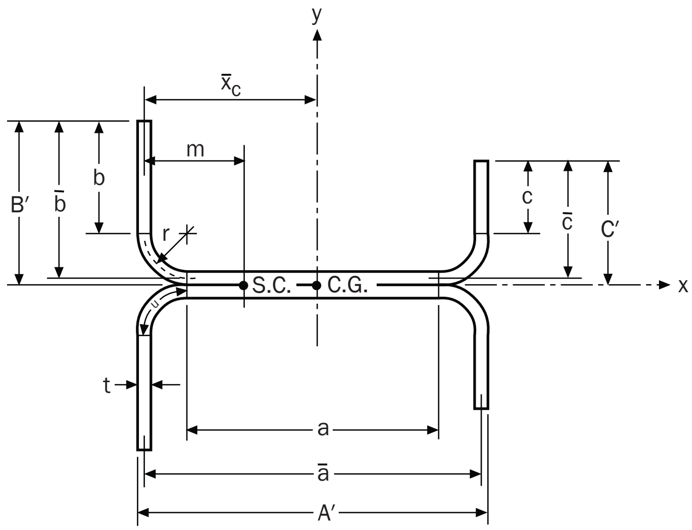


Figure 3.3.3-1
I-Section With Unequal Flanges (Singly Symmetric)

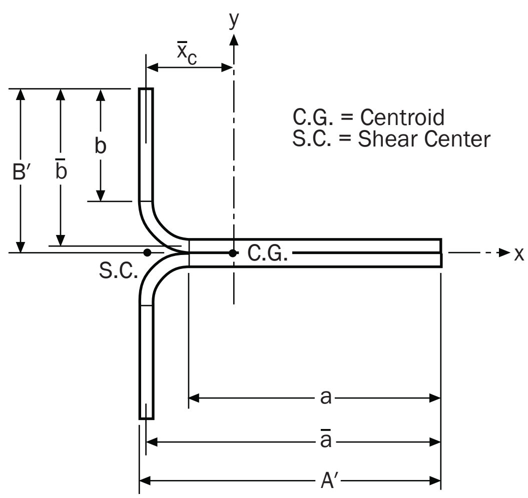


Figure 3.3.3-2
T-Section (Singly Symmetric)

1. Basic parameters

$a = A' - \left[r + t/2 + \alpha\left(r + t/2\right)\right]$ *

$\bar{a} = A' - \left(t/2 + \alpha t/2\right)$

$b = B' - \left(r + t/2\right)$

$\bar{b} = B' - t/2$

$c = \alpha\left[C' - \left(r + t/2\right)\right]$

$\bar{c} = \alpha(C' - t/2)$

$u = \pi r/2 = 1.57r$

2. Cross-sectional area

$A = t\left[2a + 2b + 2u + \alpha\left(2c + 2u\right)\right]$

* For I-Sections, α = 1.0, for T-Sections, α = 0.

3. Moment of inertia about x-axis

$$I_x = 2t\left\{\begin{array}{l} b\left(\frac{b}{2}+r+\frac{t}{2}\right)^2 + \frac{b^3}{12} + u\left(0.363r+\frac{t}{2}\right)^2 + 0.149r^3 \\ +\alpha\left[c\left(\frac{c}{2}+r+\frac{t}{2}\right)^2 + \frac{b^3}{12} + u\left(0.363r+\frac{t}{2}\right)^2 + 0.149r^3\right] \end{array}\right\}$$

4. Distance between centroid and longer flange centerline

$$\bar{x}_c = \frac{2t}{A}\left\{u0.363r + a\left(a/2+r\right) + \alpha\left[u\left(a+1.637r\right) + c\left(a+2r\right)\right]\right\}$$

5. Distance between centroid and outside of longer flange

$$\bar{x} = \bar{x}_c + \frac{t}{2}$$

6. Moment of inertia about y-axis

$$I_y = 2t\left\{\begin{array}{l} 0.358^3 + a\left(a/2+r\right)^2 + \frac{a^3}{12} \\ +\alpha\left[u\left(a+1.637r\right)^2 + 0.149r^3 + c\left(a+2r\right)^2\right] \end{array}\right\} - A\bar{x}_c^{\,2}$$

7. Distance between shear center and longer flange centerline

$$m = \bar{a}\left(1 - \frac{\bar{b}^3}{\bar{b}^3 + \bar{c}^3}\right)$$

8. Distance between shear center and centroid

$$x_o = -\left(\bar{x}_c - m\right)^*$$

9. St. Venant torsion constant

$$J = \frac{2t^3}{3}\left[a + b + u + \alpha\left(u + c\right)\right]$$

10. Warping constant
For I-Sections the value of C_w is twice the value of each channel if fastened at the middle of the webs; however, if the two channels are continuously welded at both edges of the web to form the I-Section, the warping constants are as follows:

Unlipped I-Sections and T-Sections:

$$C_w = \frac{t\bar{a}^2}{12}\left(\frac{8\bar{b}^3\bar{c}^3}{\bar{b}^3 + \bar{c}^3}\right)$$

Doubly symmetric, lipped I-Sections:
$\bar{c}$ =length of lip, see Figure 3.3.2-1

$$C_w = \frac{t\bar{b}^2}{3}\left(\bar{a}^2\bar{b} + 3\bar{a}^2\bar{c} + 6\bar{a}\,\bar{c}^2 + 4\bar{c}^3\right)$$

* Negative sign indicates x_o is measured in negative x direction.

11. Parameter used in determination of elastic critical moment

$$j = \frac{t}{2I_y}\left[\begin{array}{l} -2\bar{x}_c\bar{b}\left(\bar{x}_c^{\,2}+\bar{b}^2/3\right)+2\bar{c}\left(\bar{a}-\bar{x}_c\right)\left[\left(\bar{a}-\bar{x}_c\right)^2+\bar{c}^2/3\right] \\ +\frac{1}{2}\left[\left(\bar{a}-\bar{x}_c\right)^4-\bar{x}_c^{\,4}\right]\end{array}\right]-x_o$$

3.3.4 Z-Sections (Point-Symmetric) With and Without Lips

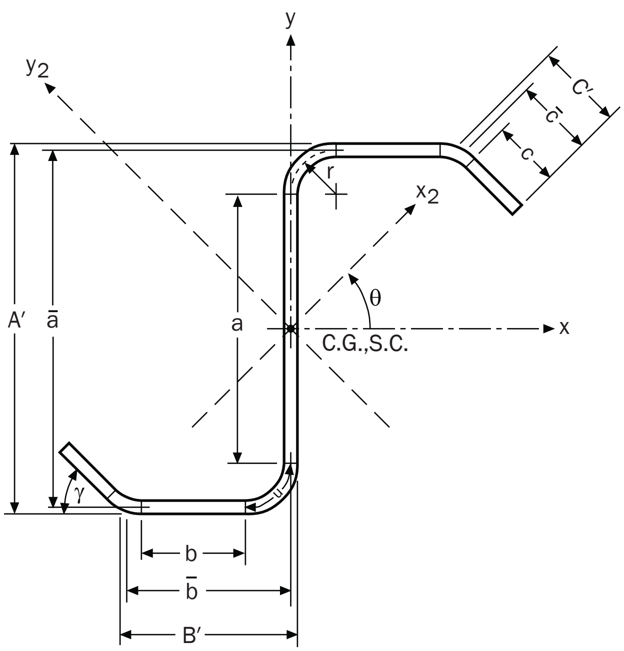


Figure 3.3.4-1
Z-Section (Point Symmetric) With Lips

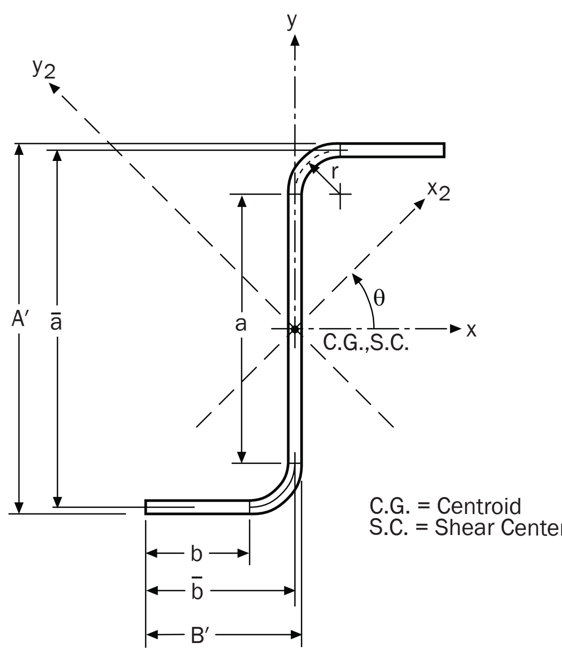


Figure 3.3.4-2
Z-Section (Point Symmetric) Without Lips

1. Basic Parameters

$a = A' - (2r + t)$

$\bar{a} = A' - t$

$b = B' - \left[r + t/2 + \alpha(r + t/2)\tan(\gamma/2)\right]$*

$\bar{b} = B' - \left(t/2 + (\alpha t/2)\tan(\gamma/2)\right)$

$c = \alpha\left[C' - (r + t/2)\tan(\gamma/2)\right]$

$\bar{c} = \alpha\left[C' - (t/2)\tan(\gamma/2)\right]$

$u_1 = \pi r/2 = 1.57r$

$u_2 = \gamma r$, where γ is in radians

2. Cross-sectional area

$A = t\left[a + 2b + 2u_1 + \alpha(2c + 2u_2)\right]$

* For sections with lips, $\alpha = 1.0$; for sections without lips, $\alpha = 0$.

3. Moment of inertia about x-axis

$$I_x = 2t\left\{\begin{array}{l} 0.0417a^3 + b(a/2+r)^2 + u_1(a/2+0.637r)^2 + 0.149r^3 \\ +\alpha\left[\begin{array}{l}\left(\dfrac{\gamma+\sin\gamma\cos\gamma}{2}-\dfrac{\sin^2\gamma}{\gamma}\right)r^3 + u_2\left(a/2+\dfrac{r\sin\gamma}{\gamma}\right)^2 \\ +\dfrac{c^3\sin^2\gamma}{12}+c\left(a/2+r\cos\gamma-\dfrac{c}{2}\sin\gamma\right)^2\end{array}\right]\end{array}\right\}$$

4. Moment of inertia about y-axis

$$I_y = 2t\left\{b(b/2+r)^2+\frac{b^3}{12}+0.356r^3+\alpha\left[\begin{array}{l} c\left(b+r(1+\sin\gamma)+\dfrac{c}{2}\cos\gamma\right)^2+\dfrac{c^3\cos^2\gamma}{12} \\ +u_2\left(b+r+\dfrac{r(1-\cos\gamma)}{\gamma}\right)^2 \\ +\left[\dfrac{\gamma-\sin\gamma\cos\gamma}{2}-\dfrac{(1-\cos\gamma)^2}{\gamma}\right]r^3\end{array}\right]\right\}$$

5. Product of inertia (See note below)

$$I_{xy} = 2t\left\{\begin{array}{l} b(a/2+r)(b/2+r)+0.5r^3+0.285ar^2 \\ +\alpha\left[\begin{array}{l} c\left(b+r(1+\sin\gamma)+\dfrac{c}{2}\cos\gamma\right)\left(\dfrac{a}{2}+r\cos\gamma-\dfrac{c}{2}\sin\gamma\right) \\ +\left(\dfrac{\sin^2\gamma}{2}+\dfrac{\sin\gamma(\cos\gamma-1)}{\gamma}\right)r^3-\dfrac{c^3\sin\gamma\cos\gamma}{12} \\ +u_2\left(b+r+\dfrac{r(1-\cos\gamma)}{\gamma}\right)\left(\dfrac{a}{2}+\dfrac{r\sin\gamma}{\gamma}\right)\end{array}\right]\end{array}\right\}$$

6. Angle between x-axis and minor principal axis, in radians (See note below)

$$\theta = \frac{\pi}{2}+0.5\arctan\left(\frac{2I_{xy}}{I_y-I_x}\right)$$

7. Moment of inertia about x_2 axis (See note below)

$$I_{x2} = I_x\cos^2\theta+I_y\sin^2\theta-2I_{xy}\sin\theta\cos\theta$$

8. Moment of inertia about y_2 axis (See note below)

$$I_{y2} = I_x\sin^2\theta+I_y\cos^2\theta+2I_{xy}\sin\theta\cos\theta$$

Note: The algebraic signs in Equations 5, 6, 7 and 8 are correct for the cross-section oriented with respect to the coordinate axes as shown in Figure 3.3.4-1 and Figure 3.3.4-2.

9. Radius of gyration about any axis

$$r = \sqrt{I/A}$$

10. Minimum radius of gyration, about x_2 axis

$$r_{min} = \sqrt{I_{x2}/A}$$

11. St. Venant torsion constant

$$J = \frac{t^3}{3}\left[a + 2b + 2u_1 + \alpha\left(2c + 2u_2\right)\right]$$

12. Warping constant

$$C_w = \frac{t}{12}\left\{\frac{\bar{a}^2\bar{b}^3\left(2\bar{a}+\bar{b}\right)+\alpha\left[\begin{array}{l}\bar{b}^2\left(4\bar{c}^4+16\bar{b}\bar{c}^3+6\bar{a}^3\bar{c}+4\bar{a}^2\bar{b}\bar{c}+8\bar{a}\bar{c}^3\right)\\ +6\bar{a}\bar{b}\bar{c}^2\left(\bar{a}+\bar{b}\right)\left(2\bar{b}\sin\gamma+\bar{a}\cos\gamma\right)\\ +4\bar{a}\bar{b}\bar{c}^3\left(2\bar{a}+4\bar{b}+\bar{c}\right)\sin\gamma\cos\gamma\\ +\bar{c}^3\left(2\bar{a}^3+4\bar{a}^2\bar{b}-8\bar{a}\bar{b}^2+\bar{a}^2\bar{c}-16\bar{b}^3-4\bar{b}^2\bar{c}\right)\cos^2\gamma\end{array}\right]}{\bar{a}+2\left(\bar{b}+\alpha\bar{c}\right)}\right\}$$

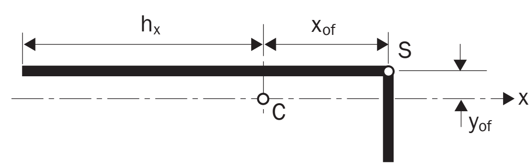
3.4 Distortional Buckling Properties

The equations below provide the flange section properties used in distortional buckling calculations in *Specification* Sections C3.1.4(b) and C4.2(b). The following is to be noted:

(1) The equations are approximations based on centerline dimensions and sharp corners. The properties calculated by this procedure are sufficiently accurate for routine engineering design of sections with small ratios of corner radius to thickness.

(2) Note that in the equations below, the *Specification* symbol x_o is presented as x_{of} to prevent confusion with similarly named symbol x_o used in *Specification* Section C3.1.2. The symbol y_o is presented as y_{of} for consistency with x_{of}

3.4.1 Flanges with 90 Degree Lips



1. Basic parameters

$$b = B' - t$$
$$d = C' - 0.5t$$

2. Flange section properties

$$A_f = \left(b + d\right)t$$

$$I_{xf} = \frac{t\left(t^2b^2 + 4bd^3 + t^2bd + d^4\right)}{12\left(b+d\right)}$$

$$I_{yf} = \frac{t\left(b^4 + 4db^3\right)}{12\left(b+d\right)}$$

$$I_{xyf} = \frac{tb^2d^2}{4\left(b+d\right)}$$

$$x_{of} = \frac{b^2}{2(b+d)}$$

$$y_{of} = \frac{-d^2}{2(b+d)}$$

$$h_x = \frac{-(b^2+2db)}{2(b+d)}$$

$$J_f = \frac{bt^3+dt^3}{3}$$

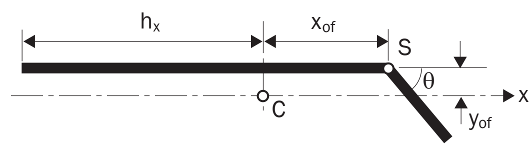
$$C_{wf} = 0$$

3.4.2 Flanges with Sloped Lips

1. Basic parameters

$b = B' - t$

$d = C' - 0.5t$



2. Flange section properties

$$A_f = (b+d)t$$

$$I_{xf} = \frac{t\left(t^2b^2+4bd^3-4bd^3\cos^2\theta+t^2bd+d^4-d^4\cos^2\theta\right)}{12(b+d)}$$

$$I_{yf} = \frac{t\left(b^4+4db^3+6d^2b^2\cos\theta+4d^3b\cos^2\theta+d^4\cos^2\theta\right)}{12(b+d)}$$

$$I_{xyf} = \frac{tbd^2\sin\theta(b+d\cos\theta)}{4(b+d)}$$

$$x_{of} = \frac{b^2-d^2\cos\theta}{2(b+d)}$$

$$y_{of} = \frac{-d^2\sin\theta}{2(b+d)}$$

$$h_x = \frac{-(b^2+2db+d^2\cos\theta)}{2(b+d)}$$

$$J_f = \frac{bt^3+dt^3}{3}$$

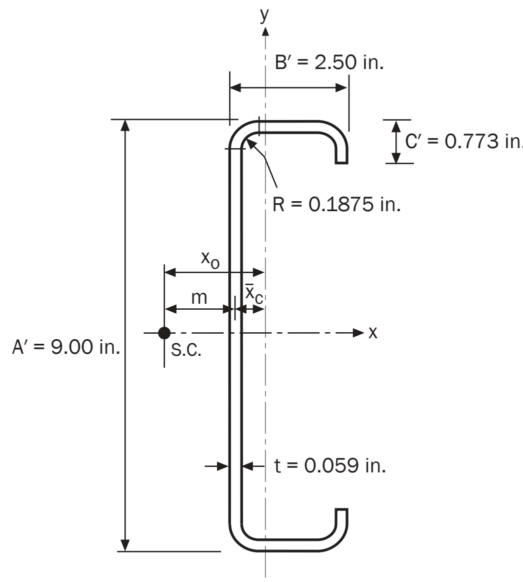
$$C_{wf} = 0$$

3.5 Gross Section Properties - Example Problems

The following example problems are intended to illustrate the use of the gross section property equations presented in this Part of the *Design Manual*. These should be used in conjunction with the other parts of the *Design Manual*.

As a general rule, section properties are computed to three significant figures. In some cases, where the properties are used in subsequent calculations, the properties are calculated to four significant figures to preserve precision. Dimensions are generally given to the nearest one thousandth of an inch. In some cases it was impractical to adhere strictly to these guidelines. Slight discrepancies should be expected between the calculated section properties computed in the examples and those given in the tables in Parts I, II and III of this *Manual* which were calculated by computer.

Example I-1: C-Section With Lips - Gross Section Properties



Given:

1. Section: 9CS2.5x059 as shown above

Required:

1. Axial and flexural properties
2. Torsional properties

Solution:

1. Axial and flexural properties

a. Basic Parameters

$A' = 9.000$ in.

$B' = 2.500$ in.

$C' = 0.773$ in.

$t = 0.059$ in.

$R = 0.1875$ in.

$\alpha = 1.0$ (section has stiffener lips)

$$r = R + t/2$$
$$= 0.1875 + 0.059/2 = 0.217 \text{ in.}$$

$$a = A' - (2r + t)$$
$$= 9.000 - [(2)(0.217) + 0.059] = 8.507 \text{ in.}$$

$$\bar{a} = A' - t$$
$$= 9.000 - 0.059 = 8.941 \text{ in.}$$

$$b = B' - [r + t/2 + \alpha(r + t/2)]$$
$$= 2.500 - [0.217 + 0.059/2 + 1.0(0.217 + 0.059/2)] = 2.007 \text{ in.}$$

$$\bar{b} = B' - [t/2 + \alpha t/2]$$
$$= 2.500 - [0.059/2 + (1.0)(0.059/2)] = 2.441 \text{ in.}$$

$$c = \alpha\left[C' - (r + t/2)\right]$$
$$= 1.0\left[0.773 - (0.217 + 0.059/2)\right] = 0.527 \text{ in.}$$
$$\bar{c} = \alpha\left[C' - (t/2)\right]$$
$$= 1.0\left[0.773 - (0.059/2)\right] = 0.744 \text{ in.}$$
$$u = \pi r/2$$
$$= \pi(0.217)/2 = 0.341 \text{ in.}$$

b. Cross-section area

$$A = t\left[a + 2b + 2u + \alpha(2c + 2u)\right]$$
$$= 0.059\left[8.507 + (2)(2.007) + (2)(0.341) + 1.0\left((2)(0.527) + (2)(0.341)\right)\right]$$
$$= 0.881 \text{ in.}^2$$

c. Moment of inertia about the x-axis

$$I_x = 2t\left\{\begin{array}{l} 0.0417a^3 + b(a/2 + r)^2 + u(a/2 + 0.637r)^2 + 0.149r^3 \\ +\alpha\left[0.0833c^3 + \dfrac{c}{4}(a - c)^2 + u\left(\dfrac{a}{2} + 0.637r\right)^2 + 0.149r^3\right] \end{array}\right\}$$

$$= (2)(0.059)\left\{\begin{array}{l} 0.0417(8.507)^3 + 2.007(8.507/2 + 0.217)^2 \\ +0.341\left[8.507/2 + 0.637(0.217)\right]^2 + 0.149(0.217)^3 \\ +1.0\left[\begin{array}{l} (0.0833)(0.527)^3 + \dfrac{0.527}{4}(8.507 - 0.527)^2 \\ +0.341\left(8.507/2 + (0.637)(0.217)\right)^2 + (0.149)(0.217)^3 \end{array}\right] \end{array}\right\}$$

$$= 0.118\left\{25.67 + 40.11 + 6.577 + 0.0015 + 1.0\left[0.012 + 8.390 + 6.577 + 0.0015\right]\right\}$$
$$= 10.3 \text{ in.}^4$$

d. Distance between centroid and web centerline

$$\bar{x}_c = \frac{2t}{A}\left\{b(b/2 + r) + u(0.363r) + \alpha\left[u(b + 1.637r) + c(b + 2r)\right]\right\}$$

$$= \frac{(2)(0.059)}{0.881}\left\{\begin{array}{l} 2.007(2.007/2 + 0.217) + (0.341)(0.363)(0.217) \\ +1.0\left[0.341\left(2.007 + (1.637)(0.217)\right) + 0.527\left(2.007 + (2)(0.217)\right)\right] \end{array}\right\}$$

$$= 0.1339\left\{2.450 + 0.0269 + 1.0\left[0.806 + 1.286\right]\right\}$$
$$= 0.612 \text{ in.}$$

e. Moment of inertia about the y-axis

$$I_y = 2t\left\{\begin{array}{l} b(b/2 + r)^2 + b^3/12 + 0.356r^3 \\ +\alpha\left[c(b + 2r)^2 + u(b + 1.637r)^2 + 0.149r^3\right] \end{array}\right\} - A\bar{x}_c^{\,2}$$

$$I_y = (2)(0.059)\left\{\begin{array}{l} 2.007\left(\dfrac{2.007}{2} + 0.217\right)^2 + \dfrac{(2.007)^3}{12} + 0.356(0.217)^3 \\ +1.0\left[\begin{array}{l} 0.527\left(2.007 + (2)(0.217)\right)^2 + \\ 0.341\left(2.007 + (1.637)(0.217)\right)^2 + 0.149(0.217)^3 \end{array}\right] \end{array}\right\} - (0.881)(0.612)^2$$

$$= 0.118\left\{2.990 + 0.674 + 0.0036 + 1.0\left[3.140 + 1.903 + 0.0015\right]\right\} - 0.330$$

I_y = 0.698 in.^{4}

f. Distance between shear center and web centerline

$$m = \bar{b}\left[\frac{3\bar{a}^2\bar{b}+\alpha\bar{c}\left(6\bar{a}^2-8\bar{c}^2\right)}{\bar{a}^3+6\bar{a}^2\bar{b}+\alpha\bar{c}\left(8\bar{c}^2-12\overline{ac}+6\bar{a}^2\right)}\right]$$

$$= 2.441\left[\frac{(3)(8.941)^2(2.441)+(1.0)(0.744)\left[(6)(8.941)^2-(8)(0.744)^2\right]}{\begin{array}{c}(8.941)^3+(6)(8.941)^2(2.441)\\+(1.0)(0.744)\left[(8)(0.744)^2-(12)(8.941)(0.744)+(6)(8.941)^2\right]\end{array}}\right]$$

$$= 2.441\left[\frac{585.4+353.6}{714.8+1171+300.8}\right]$$

= 1.048 in.

g. Distance between centroid and shear center

$$x_o = -\left(\bar{x}_c+m\right)$$

$$= -\left(0.612+1.048\right)$$

= – 1.660 in.

2. Torsional properties

a. St. Venant torsional constant

$$J = \frac{t^3}{3}\left[a+2b+2u+\alpha\left(2c+2u\right)\right]$$

$$= \frac{(0.059)^3}{3}\left[8.507+(2)(2.007)+(2)(0.341)+1.0\left((2)(0.527)+(2)(0.341)\right)\right]$$

= 0.00102 in.^{4}

b. Warping constant

$$C_w = \frac{\bar{a}^2\bar{b}^2t}{12}\left\{\frac{2\bar{a}^3\bar{b}+3\bar{a}^2\bar{b}^2+\alpha\left[\begin{array}{l}48\bar{c}^4+112\bar{b}\bar{c}^3+8\overline{ac}^3+48\bar{a}\bar{b}\bar{c}^2\\+12\bar{a}^2\bar{c}^2+12\bar{a}^2\bar{b}\bar{c}+6\bar{a}^3\bar{c}\end{array}\right]}{6\bar{a}^2\bar{b}+\left(\bar{a}+\alpha 2\bar{c}\right)^3-\alpha 24\overline{ac}^2}\right\}$$

C_w =

$$\frac{(8.941)^2(2.441)^2(0.059)}{12}\left\{\frac{\begin{array}{l}(2)(8.941)^3(2.441)+(3)(8.941)^2(2.441)^2\\ +1.0\left[\begin{array}{l}(48)(0.744)^4+(112)(2.441)(0.744)^3\\ +(8)(8.941)(0.744)^3\\ +(48)(8.941)(2.441)(0.744)^2+(12)(8.941)^2(0.744)^2\\ +(12)(8.941)^2(2.441)(0.744)+(6)(8.941)^3(0.744)\end{array}\right]\end{array}}{\begin{array}{c}(6)(8.941)^2(2.441)+(8.941+(1.0)(2)(0.744))^3\\ -(1.0)(24)(8.941)(0.744)^2\end{array}}\right\}$$

$$= 2.342\left\{\frac{3489+1429+1.0\left[\begin{array}{l}14.71+112.6+29.46+579.9\\ +531.0+1742+3191\end{array}\right]}{1171+1134-118.8}\right\}$$

= 11.9 in.^{6}

c. Parameter used in determination of elastic critical moment

$$\beta_w = -\left[\frac{t\bar{x}_c\bar{a}^3}{12}+t\bar{x}_c^{\,3}\bar{a}\right]$$

$$= -\left[\frac{(0.059)(0.612)(8.941)^3}{12}+(0.059)(0.612)^3(8.941)\right]$$

= –2.272 in.^{5}

$$\beta_f = \frac{t}{2}\left[(\bar{b}-\bar{x}_c)^4-\bar{x}_c^{\,4}\right]+\frac{t\bar{a}^2}{4}\left[(\bar{b}-\bar{x}_c)^2-\bar{x}_c^{\,2}\right]$$

$$= \frac{0.059}{2}\left[(2.441-0.612)^4-(0.612)^4\right]+\frac{(0.059)(8.941)^2}{4}\left[(2.441-0.612)^2-0.612^2\right]$$

= 3.829 in.^{5}

$$\beta_l = \alpha\left\{2\bar{c}t(\bar{b}-\bar{x}_c)^3+\frac{2}{3}t(\bar{b}-\bar{x}_c)\left[(\bar{a}/2)^3-(\bar{a}/2-\bar{c})^3\right]\right\}$$

$$=1.0\left\{\begin{array}{l}(2)(0.744)(0.059)(2.441-0.612)^3\\ +\frac{2}{3}(0.059)(2.441-0.612)\left[\left(\frac{8.941}{2}\right)^3-\left(\frac{8.941}{2}-0.744\right)^3\right]\end{array}\right\}$$

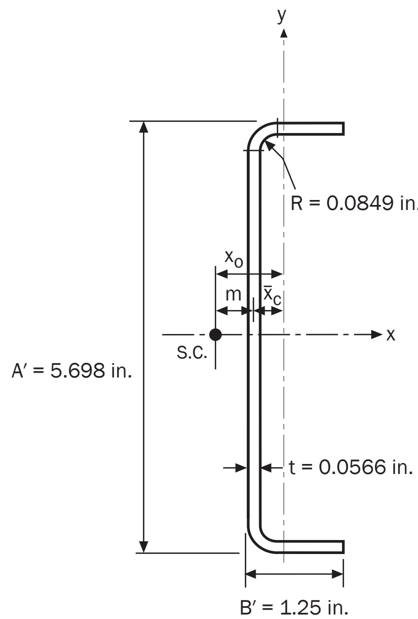
= 3.242 in.^{5}

$$j = \frac{1}{2I_y}\left(\beta_w + \beta_f + \beta_l\right) - x_o$$

$$= \frac{1}{(2)(0.698)}\left(-2.272 + 3.829 + 3.242\right) - \left(-1.660\right)$$

$$= 5.10 \text{ in.}$$

Example I-2: C-Section Without Lips - Gross Section Properties



Given:

1. Section: SSMA Track 550T125-54 as shown above

Required:

1. Axial and flexural properties
2. Torsional properties

Solution:

1. Axial and flexural properties

a. Basic Parameters

$A' = 5.698$ in.

$B' = 1.250$ in.

$C' = 0.000$ in.

$t = 0.0566$ in.

$R = 0.0849$ in.

$\alpha = 0.0$ (section does not have stiffener lips)

$r = R + t/2 = 0.0849 + 0.0566/2 = 0.113\text{ in.}$

$a = A' - (2r + t)$

$= 5.698 - [(2)(0.113) + 0.0566] = 5.415\text{ in.}$

$\bar{a} = A' - t$

$= 5.698 - 0.0566 = 5.641\text{ in.}$

$b = B' - [r + t/2 + \alpha(r + t/2)]$

$= 1.250 - [0.113 + 0.0566/2 + 0.0] = 1.109\text{ in.}$

$\bar{b} = B' - [t/2 + \alpha t/2]$

$= 1.250 - [0.0566/2 + 0.0] = 1.222\text{ in.}$

$c = \alpha[C' - (r + t/2)] = 0.0\text{ in.}$

$$\bar{c} = \alpha\left[C' - (t/2)\right] = 0.0 \text{ in.}$$

$$u = \pi r/2$$

$$= \pi(0.113)/2 = 0.177 \text{ in.}$$

b. Cross-section area

$$A = t\left[a + 2b + 2u + \alpha(2c + 2u)\right]$$

$$= 0.0566\left[5.415 + (2)(1.109) + (2)(0.177) + 0.0\right]$$

$$= 0.452 \text{ in.}^2$$

c. Moment of inertia about the x-axis

$$I_x = 2t\left\{\begin{array}{l} 0.0417a^3 + b(a/2 + r)^2 + u(a/2 + 0.637r)^2 + 0.149r^3 \\ +\alpha\left[0.0833c^3 + \dfrac{c}{4}(a - c)^2 + u\left(\dfrac{a}{2} + 0.637r\right)^2 + 0.149r^3\right] \end{array}\right\}$$

$$= (2)(0.0566)\left\{\begin{array}{l} (0.0417)(5.415)^3 + 1.109(5.415/2 + 0.113)^2 \\ +0.177\left[5.415/2 + 0.637(0.113)\right]^2 + 0.149(0.113)^3 + 0.0 \end{array}\right\}$$

$$= 1.90 \text{ in.}^4$$

d. Distance between centroid and web centerline

$$\bar{x}_c = \frac{2t}{A}\left\{b(b/2 + r) + u(0.363r) + \alpha\left[u(b + 1.637r) + c(b + 2r)\right]\right\}$$

$$= \frac{(2)(0.0566)}{0.452}\left\{1.109(1.109/2 + 0.113) + (0.177)(0.363)(0.113) + 0.0\right\}$$

$$= 0.187 \text{ in.}$$

e. Moment of inertia about the y-axis

$$I_y = 2t\left\{\begin{array}{l} b(b/2 + r)^2 + b^3/12 + 0.356r^3 \\ +\alpha\left[c(b + 2r)^2 + u(b + 1.637r)^2 + 0.149r^3\right] \end{array}\right\} - A\bar{x}_c^{\,2}$$

$$= (2)(0.0566)\left\{\begin{array}{l} 1.109(1.109/2 + 0.113)^2 + (1.109)^3/12 + 0.356(0.113)^3 \\ +0.0 \end{array}\right\} - (0.452)(0.187)^2$$

$$= 0.0531 \text{ in.}^4$$

f. Distance between shear center and web centerline

$$m = \bar{b}\left[\frac{3\bar{a}^2\bar{b} + \alpha\bar{c}\left(6\bar{a}^2 - 8\bar{c}^2\right)}{\bar{a}^3 + 6\bar{a}^2\bar{b} + \alpha\bar{c}\left(8\bar{c}^2 - 12\overline{ac} + 6\bar{a}^2\right)}\right]$$

$$= 1.222\left[\frac{(3)(5.641)^2(1.222) + 0.0}{(5.641)^3 + (6)(5.641)^2(1.222) + 0.0}\right]$$

$$= 0.345 \text{ in.}$$

g. Distance between centroid and shear center

$$x_o = -(\bar{x}_c + m)$$

$$= -(0.187 + 0.345) = -0.532 \text{ in.}$$

2. Torsional properties

a. St. Venant torsional constant

$$J = \frac{t^3}{3}\left[a + 2b + 2u + \alpha\left(2c + 2u\right)\right]$$

$$= \frac{(0.0566)^3}{3}\left[5.415 + (2)(1.109) + (2)(0.177) + 0.0\right]$$

$= 0.000483 \text{ in.}^4$

b. Warping constant

$$C_w = \frac{\bar{a}^2\bar{b}^2 t}{12}\left\{\frac{2\bar{a}^3\bar{b} + 3\bar{a}^2\bar{b}^2 + \alpha\begin{bmatrix}48\bar{c}^4 + 112\bar{b}\bar{c}^3 + 8\bar{a}\bar{c}^3 + 48\bar{a}\bar{b}\bar{c}^2 \\ +12\bar{a}^2\bar{c}^2 + 12\bar{a}^2\bar{b}\bar{c} + 6\bar{a}^3\bar{c}\end{bmatrix}}{6\bar{a}^2\bar{b} + \left(\bar{a} + \alpha 2\bar{c}\right)^3 - \alpha 24\bar{a}\bar{c}^2}\right\}$$

$$= \frac{(5.641)^2(1.222)^2(0.0566)}{12}\left\{\frac{(25.641)^3(1.222) + (3)(5.641)^2(1.222)^2 + 0.0}{(6)(5.641)^2(1.222) + (5.641 + 0.0)^3 - 0.0}\right\}$$

$= 0.316 \text{ in.}^6$

c. Parameter used in determination of elastic critical moment

$$\beta_w = -\left[\frac{t\bar{x}_c\bar{a}^3}{12} + t\bar{x}_c{}^3\bar{a}\right]$$

$$= -\left[\frac{(0.0566)(0.187)(5.641)^3}{12} + (0.0566)(0.187)^3(5.641)\right]$$

$= -0.1604 \text{ in.}^5$

$$\beta_f = \frac{t}{2}\left[\left(\bar{b} - \bar{x}_c\right)^4 - \bar{x}_c{}^4\right] + \frac{t\bar{a}^2}{4}\left[\left(\bar{b} - \bar{x}_c\right)^2 - \bar{x}_c{}^2\right]$$

$$= \frac{0.0566}{2}\left[(1.222 - 0.187)^4 - (0.187)^4\right] + \frac{(0.0566)(5.641)^2}{4}\left[(1.222 - 0.187)^2 - 0.187^2\right]$$

$= 0.4990 \text{ in.}^5$

$$\beta_l = \alpha\left\{2\bar{c}t\left(\bar{b} - \bar{x}_c\right)^3 + \frac{2}{3}t\left(\bar{b} - \bar{x}_c\right)\left[\left(\bar{a}/2\right)^3 - \left(\bar{a}/2 - \bar{c}\right)^3\right]\right\}$$

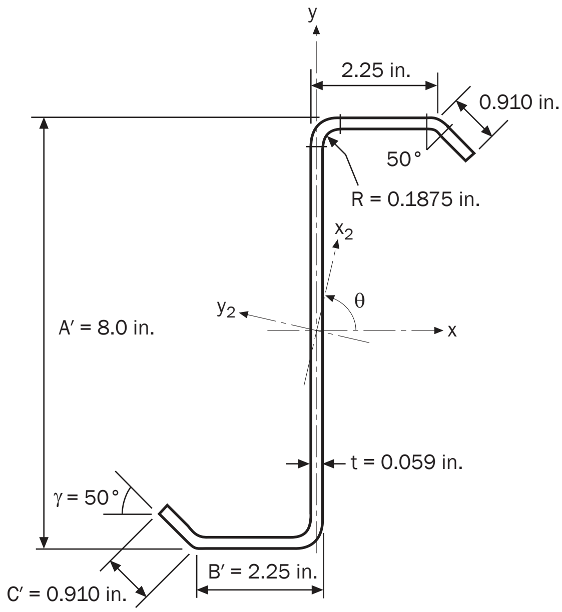
$= 0.0 \text{ in.}^5$

$$j = \frac{1}{2I_y}\left(\beta_w + \beta_f + \beta_l\right) - x_o$$

$$= \frac{1}{(2)(0.0531)}(-0.1604 + 0.4990 + 0.0) - (-0.532)$$

$= 3.72 \text{ in.}$

Example I-3: Z-Section With Lips - Gross Section Properties



Given:

1. Section: 8ZS2.25x059 as shown above

Required:

1. Axial and flexural properties
2. Torsional properties

Solution:

1. Axial and flexural properties

a. Basic Parameters

A′ = 8.000 in.

B′ = 2.250 in.

C′ = 0.910 in.

t = 0.059 in.

R = 0.1875 in.

α = 1.0 (section has stiffener lips)

$r = R + t/2 = 0.1875 + 0.059/2 = 0.217$ in.

$\gamma = 50\pi/180 = 0.8727$ radians

$a = A' - (2r + t)$

$= 8.000 - [(2)(0.217) + 0.059] = 7.507$ in.

$\bar{a} = A' - t$

$= 8.000 - 0.059 = 7.941$ in.

$b = B' - [r + t/2 + \alpha(r + t/2)\tan(\gamma/2)]$

$= 2.250 - [0.217 + 0.059/2 + 1.0(0.217 + 0.059/2)\tan(0.8727/2)] = 1.889$ in.

$\bar{b} = B' - [t/2 + (\alpha t/2)\tan(\gamma/2)]$

$= 2.250 - [0.059/2 + (1.0)(0.059/2)\tan(0.8727/2)] = 2.207$ in.

$$c = \alpha\left[C' - (r + t/2)\tan(\gamma/2)\right]$$
$$= 1.0\left[0.910 - (0.217 + 0.059/2)\tan(0.8727/2)\right] = 0.795 \text{ in.}$$
$$\bar{c} = \alpha\left[C' - (t/2)\tan(\gamma/2)\right]$$
$$= 1.0\left[0.910 - (0.059/2)\tan(0.8727/2)\right] = 0.896 \text{ in.}$$
$$u_1 = \pi r/2$$
$$= \pi(0.217)/2 = 0.341 \text{ in.}$$
$$u_2 = \gamma r$$
$$= (0.8727)(0.217) = 0.189 \text{ in.}$$

b. Cross-section area

$$A = t\left[a + 2b + 2u_1 + \alpha(2c + 2u_2)\right]$$
$$= 0.059\left[7.507 + (2)(1.889) + (2)(0.341) + 1.0\{(2)(0.795) + (2)(0.189)\}\right]$$
$$= 0.822 \text{ in.}^2$$

c. Moment of inertia about the x-axis

$$I_x = 2t\left\{\begin{array}{l} 0.0417a^3 + b(a/2+r)^2 + u_1(a/2+06.37r)^2 + 0.149r^3 \\ +\alpha\left[\begin{array}{l}\left(\dfrac{\gamma+\sin\gamma\cos\gamma}{2} - \dfrac{\sin^2\gamma}{\gamma}\right)r^3 + u_2\left(a/2 + \dfrac{r\sin\gamma}{\gamma}\right)^2 \\ +\dfrac{c^3\sin^2\gamma}{12} + c\left(a/2 + r\cos\gamma - \dfrac{c}{2}\sin\gamma\right)^2\end{array}\right]\end{array}\right\}$$

$$= 2(0.059)\left\{\begin{array}{l} 0.0417(7.507)^3 + 1.889(7.507/2+0.217)^2 \\ +0.341(7.507/2 + 0.637(0.217))^2 + 0.149(0.217)^3 \\ +(1.0)\left[\begin{array}{l}\left(\dfrac{0.8727+\sin(0.8727)\cos(0.8727)}{2} - \dfrac{\sin^2(0.8727)}{0.8727}\right)(0.217)^3 \\ +0.189\left(7.507/2 + \dfrac{(0.217)\sin(0.8727)}{0.8727}\right)^2 + \dfrac{(0.795)^3\sin^2(0.8727)}{12} \\ +(0.795)\left(\dfrac{7.507}{2} + 0.217\cos(0.8727) - \dfrac{0.795}{2}\sin(0.8727)\right)^2\end{array}\right]\end{array}\right\}$$

$$= 0.118\{17.64 + 29.78 + 5.165 + 0.0015 + 0.0001 + 2.940 + 0.0246 + 10.24\}$$
$$= 7.763 \text{ in.}^4$$

d. Moment of inertia about the y axis

$$I_y = 2t\left\{b(b/2+r)^2 + \frac{b^3}{12} + 0.356r^3 + \alpha\left[\begin{array}{l} c\left(b + r(1+\sin\gamma) + \dfrac{c}{2}\cos\gamma\right)^2 + \dfrac{c^3\cos^2\gamma}{12} \\ +u_2\left(b + r + \dfrac{r(1-\cos\gamma)}{\gamma}\right)^2 \\ +\left[\dfrac{\gamma - \sin\gamma\cos\gamma}{2} - \dfrac{(1-\cos\gamma)^2}{\gamma}\right]r^3\end{array}\right]\right\}$$

$$I_y=(2)(0.059)\left\{\begin{array}{l}1.889(1.889/2+0.217)^2+(1.889)^3/3+0.356(0.217)^3\\ +(1.0)\left[\begin{array}{l}0.795\left(1.889+0.217(1+\sin(0.8727))+\dfrac{0.795}{2}\cos(0.8727)\right)^2\\ +\dfrac{(0.795)^3\cos^2(0.8727)}{12}\\ +0.189\left(1.889+0.217+\dfrac{0.217(1-\cos(0.8727))}{0.8727}\right)^2\\ +\left(\dfrac{0.8727-\sin(0.8727)\cos(0.8727)}{2}-\dfrac{(1-\cos(0.8727))^2}{0.8727}\right)(0.217)^3\end{array}\right]\end{array}\right\}$$

$$= 0.118\{2.548+0.5617+0.0036+5.080+0.0173+0.910+0.0004\}$$

$$= 1.076 \text{ in.}^4$$

e. Product of inertia

$$I_{xy} = 2t\left\{\begin{array}{l}b(a/2+r)(b/2+r)+0.5r^3+0.285ar^2\\ +\alpha\left[\begin{array}{l}c\left(b+r(1+\sin\gamma)+\dfrac{c}{2}\cos\gamma\right)\left(\dfrac{a}{2}+r\cos\gamma-\dfrac{c}{2}\sin\gamma\right)\\ +\left(\dfrac{\sin^2\gamma}{2}+\dfrac{\sin\gamma(\cos\gamma-1)}{\gamma}\right)r^3-\dfrac{c^3\sin\gamma\cos\gamma}{12}\\ +u_2\left(b+r+\dfrac{r(1-\cos\gamma)}{\gamma}\right)\left(a/2+\dfrac{r\sin\gamma}{\gamma}\right)\end{array}\right]\end{array}\right\}$$

$$= (2)(0.059)\left\{\begin{array}{l}1.889(7.507/2+0.217)(1.889/2+0.217)+(0.5)(0.217)^3\\ +0.285(7.507)(0.217)^2\\ +1.0\left[\begin{array}{l}0.795\left(1.889+0.217(1+\sin(0.8727))+\dfrac{0.795}{2}\cos(0.8727)\right)\\ \text{x}\left(7.507/2+0.217\cos(0.8727)-\dfrac{0.795}{2}\sin(0.8727)\right)\\ +\left(\dfrac{\sin^2(0.8727)}{2}+\dfrac{\sin(0.8727)(\cos(0.8727)-1)}{0.8727}\right)0.217^3\\ -\dfrac{(0.795)^3\sin(0.8727)\cos(0.8727)}{12}\\ +0.189\left(1.889+0.217+\dfrac{0.217(1-\cos(0.8727))}{0.8727}\right)\\ \text{x}\left(7.507/2+\dfrac{0.217\sin(0.8727)}{0.8727}\right)\end{array}\right]\end{array}\right\}$$

$$= 0.118\{8.712+0.0051+0.1007+7.211-0.0002-0.0206+1.636\}$$

$$= 2.082 \text{ in.}^4$$

f. Angle between x-axis and minor principal axis, in radians

$$\theta = \frac{\pi}{2} + \frac{1}{2}\tan^{-1}\left(\frac{2I_{xy}}{I_y - I_x}\right)$$

$$= \frac{\pi}{2} + \frac{1}{2}\tan^{-1}\left(\frac{2(2.082)}{1.076 - 7.763}\right)$$

θ = 1.292 radians = 74.0 degrees

g. Moment of inertia about x_2 axis, computed using angles in radians

$$I_{x2} = I_x \cos^2\theta + I_y \sin^2\theta - 2I_{xy}\sin\theta\cos\theta$$

$$= 7.763\cos^2(1.292) + 1.076\sin^2(1.292) - (2)(2.082)\sin(1.292)\cos(1.292)$$

$= 0.481 \text{ in.}^4$

h. Moment of inertia about y_2 axis, computed using angles in radians

$$I_{y2} = I_x \sin^2\theta + I_y \cos^2\theta + 2I_{xy}\sin\theta\cos\theta$$

$$= 7.763\sin^2(1.292) + 1.076\cos^2(1.292) + (2)(2.082)\sin(1.292)\cos(1.292)$$

$= 8.36 \text{ in.}^4$

i. Minimum radius of gyration, about x_2 axis

$$r_{min} = \sqrt{I_{x2}/A}$$

$$= \sqrt{0.481/0.822}$$

$= 0.765$ in.

2. Torsional properties

a. St. Venant torsional constant

$$J = \frac{t^3}{3}\left[a + 2b + 2u_1 + \alpha(2c + 2u_2)\right]$$

$$= \frac{0.059^3}{3}\left[7.507 + (2)(1.889) + (2)(0.341) + 1.0\left((2)(0.795) + (2)(0.189)\right)\right]$$

$= 0.000954 \text{ in.}^4$

b. Warping constant

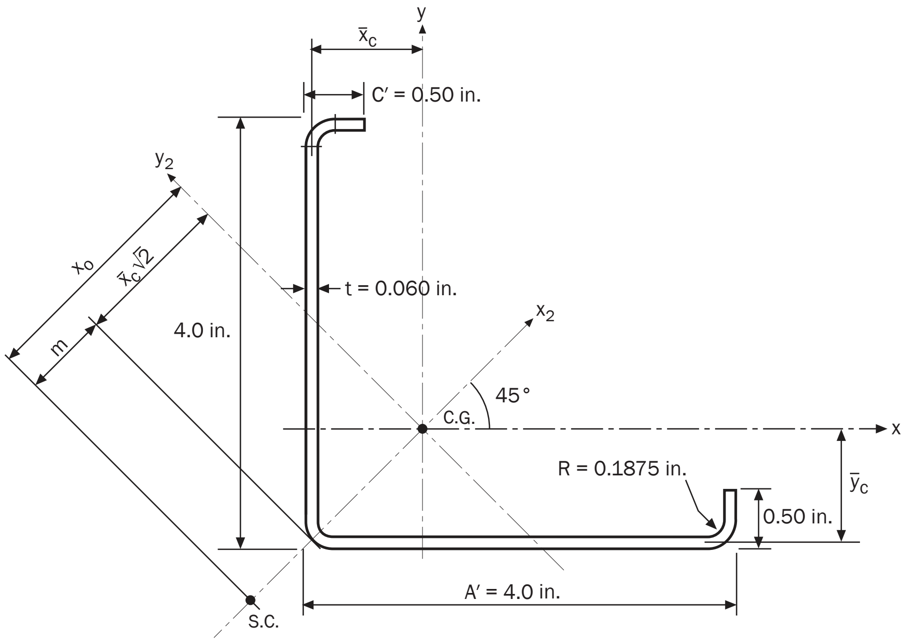
$$C_w = \frac{t}{12}\left\{\frac{\bar{a}^2\bar{b}^3\left(2\bar{a}+\bar{b}\right) + \alpha\left[\begin{array}{l}\bar{b}^2\left[4\bar{c}^4 + 16\bar{b}\bar{c}^3 + 6\bar{a}^3\bar{c} + 4\bar{a}^2\bar{b}\bar{c} + 8\bar{a}\bar{c}^3\right] \\ +6\bar{a}\bar{b}\bar{c}^2\left(\bar{a}+\bar{b}\right)\left[2\bar{b}\sin\gamma + \bar{a}\cos\gamma\right] \\ +4\bar{a}\bar{b}\bar{c}^3\left(2\bar{a}+4\bar{b}+\bar{c}\right)\sin\gamma\cos\gamma \\ +\bar{c}^3\left[2\bar{a}^3 + 4\bar{a}^2\bar{b} - 8\bar{a}\bar{b}^2 + \bar{a}^2\bar{c} - 16\bar{b}^3 - 4\bar{b}^2\bar{c}\right]\cos^2\gamma\end{array}\right]}{\bar{a} + 2\bar{b} + \alpha 2\bar{c}}\right\}$$

$$C_w = \frac{0.059}{12}\left\{\frac{\begin{array}{l}\left((7.941)^2(2.207)^3\left[(2)(7.941)+2.207\right]\right)+\\ (1.0)\left[\begin{array}{l}(2.207)^2\left[\begin{array}{l}(4)(0.896)^4+(16)(2.207)(0.896)^3\\ +(6)(7.941)^3(0.896)\\ +(4)(7.941)^2(2.207)(0.896)\\ +(8)(7.941)(0.896)^3\end{array}\right]\\ +(6)(7.941)(2.207)(0.896)^2(7.941+2.207)\left[\begin{array}{l}(2)(2.207)\sin(0.8727)\\ +(7.941)\cos(0.8727)\end{array}\right]\\ +(4)(7.941)(2.207)(0.896)^3\left[\begin{array}{l}(2)(7.941)\\ +(4)(2.207)\\ +0.896\end{array}\right]\sin(0.8727)\cos(0.8727)\\ +(0.896)^3\left[\begin{array}{l}(2)(7.941)^3+(4)(7.941)^2(2.207)\\ -(8)(7.941)(2.207)^2\\ +(7.941)^2(0.896)\\ -16(2.207)^3-4(2.207)^2(0.896)\end{array}\right]\cos^2(0.8727)\end{array}\right]\end{array}}{7.941+2(2.207)+1.0(2)(0.896)}\right\}$$

$$= \frac{0.059}{12}\left(\frac{12262+1.0(15901+7270+635.8+331.6)}{14.15}\right)$$

$$= 12.6 \text{ in.}^6$$

Example I-4: Equal Leg Angle With Lips - Gross Section Properties



Given:

1. Section: 4LS4x060 as shown above

Required:

1. Axial and flexural properties
2. Torsional properties

Solution:

1. Axial and flexural properties

a. Basic Parameters

$A' = 4.000$ in.

$C' = 0.500$ in.

$t = 0.060$ in.

$R = 0.1875$ in.

$\alpha = 1.0$ (section has stiffener lips)

$r = R + t/2 = 0.1875 + 0.060/2 = 0.218$ in.

$$a = A' - [r + t/2 + \alpha(r + t/2)]$$
$$= 4.000 - [0.218 + 0.060/2 + 1.0(0.218 + 0.060/2)] = 3.504 \text{ in.}$$

$$\bar{a} = A' - [t/2 + \alpha t/2]$$
$$= 4.000 - [0.060/2 + (1.0)(0.060)/2] = 3.940 \text{ in.}$$

$$c = \alpha[C' - (r + t/2)]$$
$$= (1.0)[0.500 - (0.218 + 0.060/2)] = 0.252 \text{ in.}$$

$$\bar{c} = \alpha[C' - t/2]$$
$$= (1.0)[0.500 - 0.060/2] = 0.470 \text{ in.}$$

$u = \pi r/2 = \pi(0.218)/2 = 0.342$ in.

b. Cross-section area

$$A = t\left[2a + u + \alpha(2c + 2u)\right]$$

$$A = 0.060\left[(2)(3.504) + 0.342 + (1.0)((2)(0.252) + (2)(0.342))\right]$$

$= 0.512$ in.^{2}

c. Distance between centroid and centerlines of webs

$$\bar{x}_c = \bar{y}_c = \frac{t}{A}\left\{a\left(\frac{a}{2} + r\right) + u(0.363r) + \alpha\left[c\left(a + \frac{c}{2} + 3r\right) + u(a + 2r)\right]\right\}$$

$$= \frac{0.060}{0.512}\left\{\begin{array}{l}(3.504)\left(\frac{3.504}{2} + 0.218\right) + (0.342)(0.363)(0.218) \\ +(1.0)\left[0.252\left(3.504 + \frac{0.252}{2} + (3)(0.218)\right) + 0.342(3.504 + (2)(0.218))\right]\end{array}\right\}$$

$= 1.097$ in.

d. Moment of inertia about x and y axes

$$I_x = I_y = t\left\{\begin{array}{l}a\left(\frac{a}{2} + r\right)^2 + \frac{a^3}{12} + u(0.363r)^2 + 0.149r^3 \\ +\alpha\left[\begin{array}{l}c(a + 2r)^2 + \frac{c^3}{12} + c\left(\frac{c}{2} + r\right)^2 + u(a + 1.637r)^2 \\ +u(0.363r)^2 + (2)(0.149)r^3\end{array}\right]\end{array}\right\} - A\bar{x}_c^2$$

$$= 0.060\left\{\begin{array}{l}3.504\left(\frac{3.504}{2} + 0.218\right)^2 + \frac{(3.504)^3}{12} \\ +(0.342)\left[(0.363)(0.218)\right]^2 + (0.149)(0.218)^3 \\ +1.0\left[\begin{array}{l}0.252\left[3.504 + (2)(0.218)\right]^2 + \frac{(0.252)^3}{12} \\ +0.252\left(\frac{0.252}{2} + 0.218\right)^2 \\ +0.342\left[3.504 + (1.637)(0.218)\right]^2 \\ +0.342\left[0.363(0.218)\right]^2 + (2)(0.149)(0.218)^3\end{array}\right]\end{array}\right\} - (0.512)(1.097)^2$$

$= 0.958$ in.^{4}

e. Product of inertia

$$I_{xy} = t\left\{\begin{array}{l}-0.137r^3 + u(0.363r)^2 \\ +2\alpha\left[c(a + 2r)\left(\frac{c}{2} + r\right) + 0.137r^3 + u(a + 1.637r)(0.363r)\right]\end{array}\right\} - A\bar{x}_c\bar{y}_c$$

I_{xy} =

$$0.060\left\{\begin{array}{l} -0.137(0.218)^3+0.342\left[0.363(0.218)\right]^2 \\ +(2)(1.0)\left[\begin{array}{l} 0.252\left[3.504+2(0.218)\right]\left(\dfrac{0.252}{2}+0.218\right) \\ +0.137(0.218)^3 \\ +0.342\left[3.504+1.637(0.218)\right](0.363)(0.218) \end{array}\right] \end{array}\right\}-(0.512)(1.097)(1.097)$$

$$= 0.060\left\{-0.00142+0.00214+(2.0)\left[0.3416+0.00142+0.1045\right]\right\}-0.6161$$

= –0.562 in.^{4}

f. Moment of inertia about y_2 axis

I_{y2} = $I_x + I_{xy}$

$$= 0.958+(-0.562)=0.396 \text{ in.}^4$$

2. Torsional properties

a. Distance between shear center and centerline of square corner

$$m = \frac{\bar{a}\bar{c}^2\sqrt{2}}{2}\frac{(3\bar{a}-2\bar{c})}{\left[2\bar{a}^3-(\bar{a}-\bar{c})^3\right]}$$

$$= \frac{(3.940)(0.470)^2\sqrt{2}}{2}\frac{\left[(3)(3.940)-(2)(0.470)\right]}{\left[(2)(3.940)^3-(3.940-0.470)^3\right]} = 0.083 \text{ in.}$$

b. St. Venant torsion constant

$$J = \frac{t^3}{3}\left[2a+u+\alpha(2c+2u)\right]$$

$$= \frac{0.060^3}{3}\left[(2)(3.504)+0.342+1.0\left(2(0.252)+2(0.342)\right)\right] = 0.000615 \text{ in.}^4$$

c. Warping constant

$$C_w = \frac{\bar{a}^4\bar{c}^3t}{6}\frac{(4\bar{a}+3\bar{c})}{\left[2\bar{a}^3-(\bar{a}-\bar{c})^3\right]}$$

$$= \frac{(3.940)^4(0.470)^3(0.060)}{6}\frac{(4)(3.940)+(3)(0.470)}{\left[(2)(3.940)^3-(3.940-0.470)^3\right]} = 0.0533 \text{ in.}^6$$

d. Distance from centroid to shear center

$$x_o = -\left(\bar{x}_c\sqrt{2}+m\right)$$

$$= -\left(1.097\sqrt{2}+0.083\right) = -1.634 \text{ in.}$$

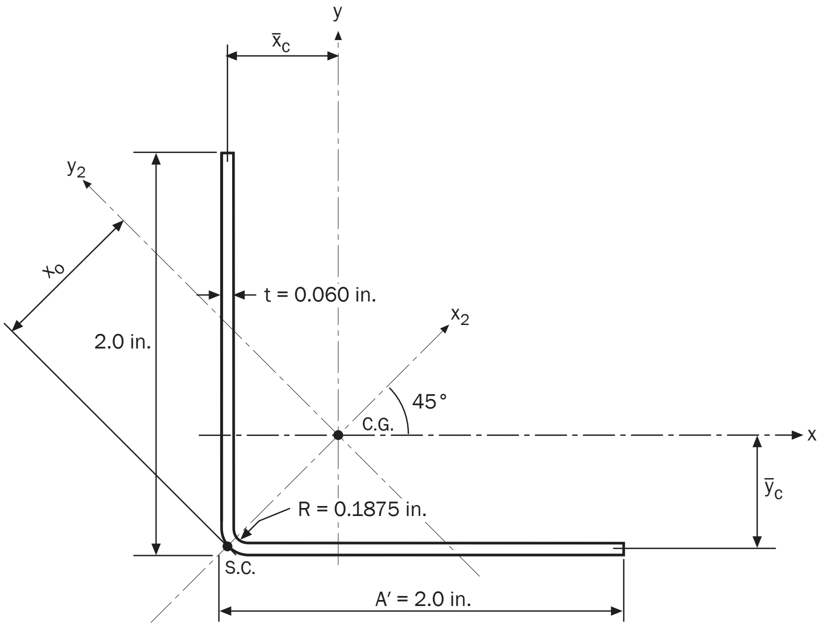
e. Parameter used to determine elastic critical moment

$$j = \frac{\sqrt{2}t}{48I_{y2}}\left[\bar{a}^4+4\bar{a}^3\bar{c}-6\bar{a}^2\bar{c}^2+\bar{c}^4\right]-x_o$$

$$= \frac{\sqrt{2}(0.060)}{(48)(0.396)}\left[\begin{array}{l}(3.940)^4+(4)(3.940)^3(0.470) \\ -(6)(3.940)^2(0.470)^2+(0.470)^4\end{array}\right]-(-1.634)$$

= 3.13 in.

Example I-5: Equal Leg Angle Without Lips - Gross Section Properties



Given:

1. Section 2LU2x060 as shown above

Required:

1. Axial and flexural properties
2. Torsional properties

Solution:

1. Axial and flexural properties

a. Basic Parameters:

$A' = 2.000$ in.

$C' = 0.000$ in.

$t = 0.060$ in.

$R = 0.1875$ in.

$\alpha = 0.0$ (section does not have stiffener lips)

$r = R + t/2 = 0.1875 + 0.060/2 = 0.218$ in.

$$a = A' - \left[r + t/2 + a\left(r + t/2\right)\right]$$
$$= 2.000 - \left[0.218 + 0.060/2 + 0.0\right] = 1.752 \text{ in.}$$

$$\bar{a} = A' - \left[t/2 + \alpha t/2\right]$$
$$= 2.000 - \left[0.060/2 + 0.0\right] = 1.970 \text{ in.}$$

$$c = \alpha\left[C' - \left(r + t/2\right)\right]$$
$$= 0.0 \text{ in.}$$

$$\bar{c} = \alpha\left[C' - \left(t/2\right)\right]$$
$$= 0.0 \text{ in.}$$

$$u = \pi r/2$$
$$= \pi\left(0.218\right)/2 = 0.342 \text{ in.}$$

b. Cross-section area

$$A = t\left[2a + u + \alpha(2c + 2u)\right]$$
$$= 0.060\left[(2)(1.752) + 0.342 + 0.0\right]$$
$$= 0.231 \text{ in.}^2$$

c. Distance between centroid and centerlines of webs

$$\bar{x}_c = \bar{y}_c = \frac{t}{A}\left\{a\left(\frac{a}{2} + r\right) + u(0.363r) + \alpha\left[c\left(a + \frac{c}{2} + 3r\right) + u(a + 2r)\right]\right\}$$
$$= \frac{0.060}{0.231}\left\{(1.752)\left(\frac{1.752}{2} + 0.218\right) + (0.342)(0.363)(0.218) + 0.0\right\}$$
$$= 0.505 \text{ in.}$$

d. Moment of inertia about x and y axes

$$I_x = I_y = t\left\{\begin{array}{l} a\left(\frac{a}{2} + r\right)^2 + \frac{a^3}{12} + u(0.363r)^2 + 0.149r^3 \\ +\alpha\left[\begin{array}{l} c(a + 2r)^2 + \frac{c^3}{12} + c\left(\frac{c}{2} + r\right)^2 + u(a + 1.637r)^2 \\ +u(0.363r)^2 + (2)(0.149)r^3 \end{array}\right] \end{array}\right\} - A\bar{x}_c^2$$
$$= 0.060\left\{\begin{array}{l} 1.752\left(\frac{1.752}{2} + 0.218\right)^2 + \frac{(1.752)^3}{12} \\ +(0.342)\left[(0.363)(0.218)\right]^2 + (0.149)(0.218)^3 + 0.0 \end{array}\right\} - (0.231)(0.505)^2$$
$$= 0.0940 \text{ in.}^4$$

e. Product of inertia

$$I_{xy} = t\left\{\begin{array}{l} -0.137r^3 + u(0.363r)^2 \\ +2\alpha\left[c(a + 2r)\left(\frac{c}{2} + r\right) + 0.137r^3 + u(a + 1.637r)(0.363r)\right] \end{array}\right\} - A\bar{x}_c\bar{y}_c$$
$$= 0.060\left\{-0.137(0.218)^3 + 0.342\left[0.363(0.218)\right]^2 + 0.0\right\} - (0.231)(0.505)(0.505)$$
$$= -0.0589 \text{ in.}^4$$

f. Moment of inertia about y_2 axis

$$I_{y2} = I_x + I_{xy}$$
$$= 0.0940 + (-0.0589)$$
$$= 0.0351 \text{ in.}^4$$

2. Torsional properties

a. Distance between shear center and centerline of square corner

$$m = \frac{\bar{a}\bar{c}^2\sqrt{2}}{2}\frac{(3\bar{a} - 2\bar{c})}{\left[2\bar{a}^3 - (\bar{a} - \bar{c})^3\right]}$$

$$m = 0.000 \text{ in.}$$

b. St. Venant torsional constant

$$J = \frac{t^3}{3}\left[2a + u + \alpha\left(2c + 2u\right)\right]$$

$$= \frac{(0.060)^3}{3}\left[(2)(1.752) + 0.342 + 0.0\right]$$

$$= 0.000277 \text{ in.}^4$$

c. Warping constant

$$C_w = \frac{\bar{a}^4\bar{c}^3 t}{6}\frac{\left(4\bar{a} + 3\bar{c}\right)}{\left[2\bar{a}^3 - \left(\bar{a} - \bar{c}\right)^3\right]}$$

$$= 0.000 \text{ in.}^6$$

d. Distance from centroid to shear center

$$x_o = -\left(\bar{x}_c\sqrt{2} + m\right)$$

$$= -\left(0.505\sqrt{2} + 0.000\right)$$

$$= -0.714 \text{ in.}$$

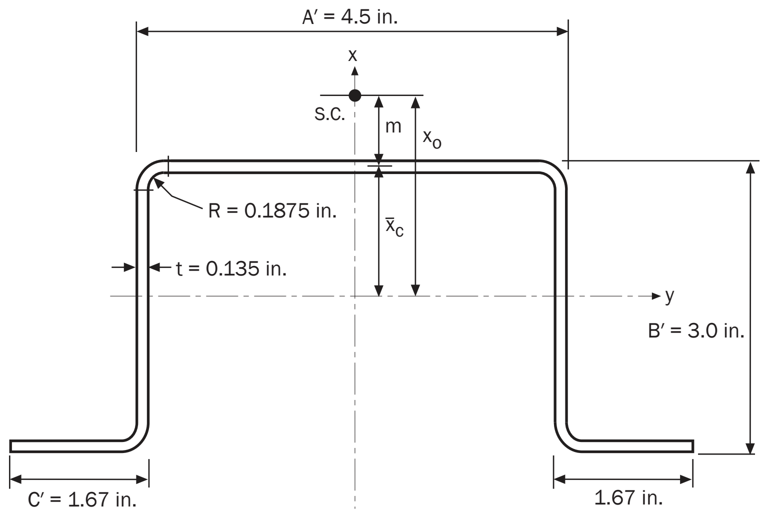
e. Parameter used to determine elastic critical moment

$$j = \frac{\sqrt{2}t}{48I_{y2}}\left[\bar{a}^4 + 4\bar{a}^3\bar{c} - 6\bar{a}^2\bar{c}^2 + \bar{c}^4\right] - x_o$$

$$= \frac{\sqrt{2}(0.060)}{(48)(0.0351)}\left[(1.970)^4 + 0.0 - 0.0 + 0.0\right] - (-0.714)$$

$$= 1.47 \text{ in.}$$

Example I-6: Hat Section without Lips - Gross Section Properties



Given:

1. Section: 3HU4.5x135 as shown above

Required:

1. Axial and flexural properties
2. Torsional properties

Solution:

1. Axial and flexural properties

a. Basic Parameters

$A' = 4.50$ in.

$B' = 3.00$ in.

$C' = 1.670$ in.

$t = 0.135$ in.

$R = 0.1875$ in.

$\alpha = 1.0$

$r = R + t/2$

$= 0.1875 + 0.135/2 = 0.255 \text{ in.}$

$a = A' - (2r + t)$

$= 4.5 - [(2)(0.255) + 0.135] = 3.855 \text{ in.}$

$\bar{a} = A' - t$

$= 4.50 - 0.135 = 4.365 \text{ in.}$

$b = B' - [r + t/2 + \alpha(2r + t)]$

$= 3.00 - [0.255 + 0.135/2 + 1.0(0.255 + 0.135/2)] = 2.355 \text{ in.}$

$\bar{b} = B' - (t/2 + \alpha t/2)$

$= 3.00 - [0.135/2 + (1.0)(0.135/2)] = 2.865 \text{ in.}$

$c = \alpha[C' - (r + t/2)]$

$= 1.0[1.670 - (0.255 + 0.135/2)] = 1.348 \text{ in.}$

$$\bar{c} = \alpha(C' - t/2)$$

$$= 1.0\left[1.670 - (0.135/2)\right] = 1.603 \text{ in.}$$

$$u = \pi r/2$$

$$= \pi(0.255)/2 = 0.401 \text{ in.}$$

b. Cross-section area

$$A = t\left[a + 2b + 2u + \alpha(2c + 2u)\right]$$

$$= 0.135\left[3.855 + (2)(2.355) + (2)(0.401) + 1.0\{(2)(1.348) + (2)(0.401)\}\right]$$

$$= 1.737 \text{ in.}^2$$

c. Moment of inertia about the x axis

$$I_x = 2t\left\{\begin{array}{l} 0.0417a^3 + b(a/2+r)^2 + u(a/2+0.637r)^2 + 0.149r^3 \\ +\alpha\left[0.0833c^3 + \dfrac{c}{4}(a+c+4r)^2 + u\left(\dfrac{a}{2}+1.363r\right)^2 + 0.149r^3\right] \end{array}\right\}$$

$$= (2)(0.135)\left\{\begin{array}{l} 0.0417(3.855)^3 + 2.355(3.855/2+0.255)^2 \\ +0.401\left[3.855/2+0.637(0.255)\right]^2 + 0.149(0.255)^3 \\ +1.0\left[\begin{array}{l}(0.0833)(1.348)^3 + \dfrac{1.348}{4}(3.855+1.348+(4)(0.255))^2 \\ +0.401(3.855/2+(1.363)(0.255))^2 + (0.149)(0.255)^3\end{array}\right] \end{array}\right\}$$

$$= 0.270\{2.389 + 11.22 + 1.752 + 0.0025 + 1.0[0.2040 + 13.05 + 2.076 + 0.0025]\}$$

$$= 8.29 \text{ in.}^4$$

d. Distance between centroid and web centerline

$$\bar{x}_c = \frac{2t}{A}\{b(b/2+r) + u(0.363r) + \alpha[u(b+1.637r) + c(b+2r)]\}$$

$$= \frac{(2)(0.135)}{1.737}\left\{\begin{array}{l} 2.355(2.355/2+0.255) + (0.401)(0.363)(0.255) \\ +1.0\left[0.401(2.355+(1.637)(0.255)) + 1.348(2.355+(2)(0.255))\right] \end{array}\right\}$$

$$= 0.1554\{3.374 + 0.0371 + 1.0\ [1.112 + 3.862]\}$$

$$= 1.303 \text{ in.}$$

e. Moment of inertia about the y axis

$$I_y = 2t\left\{\begin{array}{l} b(b/2+r)^2 + b^3/12 + 0.356r^3 \\ +\alpha\left[c(b+2r)^2 + u(b+1.637r)^2 + 0.149r^3\right] \end{array}\right\} - A\bar{x}_c^{\,2}$$

$$I_y = (2)(0.135)\left\{\begin{array}{l} 2.355(2.355/2+0.255)^2 + \dfrac{(2.355)^3}{12} + 0.356(0.255)^3 \\ +1.0\left[\begin{array}{l} 1.348(2.355+(2)(0.255))^2 \\ +0.401(2.355+1.637(0.255))^2 \\ +0.149(0.255)^3 \end{array}\right] \end{array}\right\} - (1.737)(1.303)^2$$

$$= 0.270\{4.833 + 1.088 + 0.0059 + 1.0[11.06 + 3.082 + 0.0025]\} - 2.949$$

I_y = 2.470 in.^{4}

f. Distance between shear center and web centerline

$$m = \bar{b}\left[\frac{3\bar{a}^2\bar{b} + \alpha\bar{c}\left(6\bar{a}^2 - 8\bar{c}^2\right)}{\bar{a}^3 + 6\bar{a}^2\bar{b} + \alpha\bar{c}\left[8\bar{c}^2 + 12\bar{a}\bar{c} + 6\bar{a}^2\right]}\right]$$

$$= 2.865\left[\frac{(3)(4.365)^2(2.865) + (1.0)(1.603)\left((6)(4.365)^2 - (8)(1.603)^2\right)}{\begin{array}{c}(4.365)^3 + (6)(4.365)^2(2.865) \\ +(1.0)(1.603)\left[(8)(1.603)^2 + (12)(4.365)(1.603) + (6)(4.365)^2\right]\end{array}}\right]$$

$$= 2.865\left[\frac{163.8 + 150.3}{83.17 + 327.5 + 350.8}\right]$$

= 1.182 in.

g. Distance between centroid and shear center

$$x_o = -\left(\bar{x}_c + m\right)$$

$$= -(1.303 + 1.182)$$

= –2.485 in.

2. Torsional properties

a. St. Venant torsional constant

$$J = \frac{t^3}{3}\left[a + 2b + 2u + \alpha(2c + 2u)\right]$$

$$= \frac{0.135^3}{3}\left[3.855 + (2)(2.355) + (2)(0.401) + 1.0\{(2)(1.348) + (2)(0.401)\}\right]$$

= 0.0106 in.^{4}

b. Warping constant

$$C_w = \frac{\bar{a}^2\bar{b}^2 t}{12}\left\{\frac{2\bar{a}^3\bar{b} + 3\bar{a}^2\bar{b}^2 + \alpha\left[\begin{array}{l}48\bar{c}^4 + 112\bar{b}\bar{c}^3 + 8\bar{a}\bar{c}^3 - 48\bar{a}\bar{b}\bar{c}^2 \\ -12\bar{a}^2\bar{c}^2 + 12\bar{a}^2\bar{b}\bar{c} + 6\bar{a}^3\bar{c}\end{array}\right]}{6\bar{a}^2\bar{b} + \left(\bar{a} + \alpha 2\bar{c}\right)^3}\right\}$$

C_w =

$$\frac{(4.365)^2(2.865)^2(0.135)}{12}\left\{\frac{\begin{array}{l}(2)(4.365)^3(2.865)+(3)(4.365)^2(2.865)^2\\ +1.0\left[\begin{array}{l}(48)(1.603)^4+(112)(2.865)(1.603)^3\\ +(8)(4.365)(1.603)^3\\ -(48)(4.365)(2.865)(1.603)^2-(12)(4.365)^2(1.603)^2\\ +(12)(4.365)^2(2.865)(1.603)+(6)(4.365)^3(1.603)\end{array}\right]\end{array}}{(6)(4.365)^2(2.865)+\left(4.365+(1.0)(2)(1.603)\right)^3}\right\}$$

$$= 1.759\left\{\frac{476.6+469.2+1.0\left[\begin{array}{l}316.9+1322+143.8-1543\\ -587.5+1050+799.9\end{array}\right]}{327.5+434.0}\right\}$$

$= 5.65 \text{ in.}^6$

c. Parameter used in determination of elastic critical moment

$$\beta_w = -\left[\frac{t\bar{x}_c\bar{a}^3}{12}+t\bar{x}_c^{\,3}\bar{a}\right]$$

$$= -\left[\frac{(0.135)(1.303)(4.365)^3}{12}+(0.135)(1.303)^3(4.365)\right]$$

$= -2.523 \text{ in.}^5$

$$\beta_f = \frac{t}{2}\left[\left(\bar{b}-\bar{x}_c\right)^4-\bar{x}_c^{\,4}\right]+\frac{t\bar{a}^2}{4}\left[\left(\bar{b}-\bar{x}_c\right)^2-\bar{x}_c^{\,2}\right]$$

$$= \frac{0.135}{2}\left[(2.865-1.303)^4-(1.303)^4\right]+\frac{(0.135)(4.365)^2}{4}\left[(2.865-1.303)^2-1.303^2\right]$$

$= 0.6844 \text{ in.}^5$

$$\beta_l = 2\bar{c}t\left(\bar{b}-\bar{x}_c\right)^3+\frac{2}{3}t\left(\bar{b}-\bar{x}_c\right)\left[\left(\frac{\bar{a}}{2}+\bar{c}\right)^3-\left(\frac{\bar{a}}{2}\right)^3\right]$$

$$= \left\{\begin{array}{l}(2)(1.603)(0.135)(2.865-1.303)^3\\ +\frac{2}{3}(0.135)(2.865-1.303)\left[\left(\frac{4.365}{2}+1.603\right)^3-\left(\frac{4.365}{2}\right)^3\right]\end{array}\right\}$$

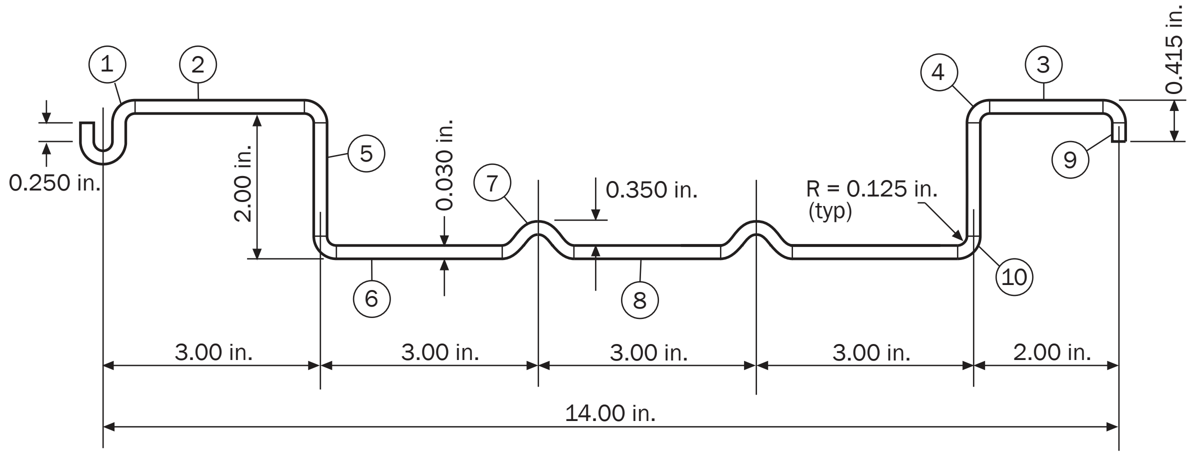
$= 7.814 \text{ in.}^5$

$$j = \frac{1}{2I_y}(\beta_w + \beta_f + \beta_l) - x_o$$

$$= \frac{1}{(2)(2.470)}(-2.523 + 0.6844 + 7.814) - (-2.485)$$

$$= 3.69 \text{ in.}$$

Example I-7: Wall Panel Section with Intermediate Stiffeners - Gross Section Properties



Given:

1. Section: Shown in sketch above

Required:

1. Gross section properties

Solution:

Since no closed formed solution is available, the properties must be determined by parts.

1. Elements 4 and 10

90° corners:

$r = R + t/2 = 0.125 + 0.030/2 = 0.140$ in.

Length of arc:

$u = 1.57r = (1.57)(0.140) = 0.220$ in.

Distance of c.g. from center of radius:

$c = 0.637r = (0.637)(0.140) = 0.089$ in.

Distance of c.g. from top of panel:

$y = 0.125 + 0.030 - 0.089 = 0.066$ in. (element 4)

$y = 2.00 + (0.125 - 0.089) = 1.964$ in. (element 10)

I'_x (each arc):

$I'_x = 0.149r^3 = (0.149)(0.140)^3 = 0.0004$ in.^{3}

2. Element 7

$r = 0.140$ in., $\theta = 45° = 0.785$ rad.

$c_1 = r\sin\theta/\theta = (0.140)(0.707)/0.785 = 0.126$ in.

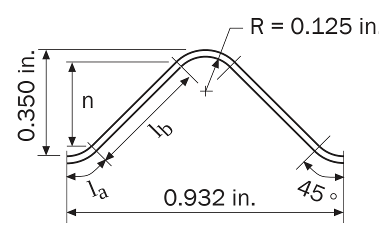
$n = 0.350 - (2)(0.140)(1 - \cos(0.785))$

$= 0.350 - 0.082 = 0.268$ in.

$l_b = 0.268/\sin(0.785) = 0.379$ in.

$l_a = \theta r = (0.785)(0.140) = 0.110$ in.

Element 7



I'_1 (straight portions):

$I'_1 = (2)(1/12)(l_b)n^2$

$= (2)(1/12)(0.379)(0.268)^2 = 0.0045$ in.^{3}

I'_1 (each arc):

$$I'_1 = \left[\frac{\theta + \sin\theta\cos\theta}{2} - \frac{\sin^2\theta}{\theta}\right] r^3$$

$$= \left[\frac{0.785 + \sin(0.785)\cos(0.785)}{2} - \frac{\sin^2(0.785)}{0.785}\right](0.140)^3$$

$= 0.000017 \text{ in.}^3 \approx 0$

By inspection, take advantage of symmetry and locate reference axis at 1/2 element.

depth = $(0.350 + 0.030)/2 = 0.190$ in.

Segment	L (in.)	y (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 about own axis (in.^{3})
Upper Radius	0.220	0.161	0.035	0.0057	----
Straight Segments	(2)(0.379) = 0.758	0.000	0.000	-----	0.0045
Lower Radii	0.220	-0.161	- 0.035	0.0057	----
Sum $\sum$	1.198		0.000	0.0114	0.0045

$y_{cg} = \sum Ly / \sum L$

$= 0.000/1.198 = 0.000$ in. (at centerline as expected)

$$\sum I'_x = \sum Ly^2 + \sum I'_1 - y_{cg}{}^2 \sum L$$

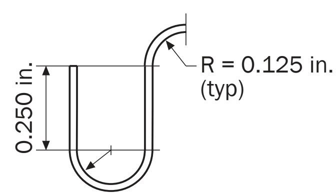
$= 0.0114 + 0.0045 - (0.000)^2 (1.198) = 0.0159 \text{ in.}^3$

Distance of c.g. from top of panel

$\bar{y} = 2.030 - (0.350 + 0.030)/2 = 1.840$ in.

3. Element 1

Element 1



Segment	L (in.)	y from top flange (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_1 about own axis (in.^{3})
90° Corner	0.220	0.155 - 0.089 = 0.066	0.015	0.0010	----
Straight Segments	(2)(0.250) = 0.500	0.280	0.140	0.0392	0.0026
Semi-Circle	(2)(0.220) = 0.440	0.405 + 0.089 = 0.494	0.217	0.1074	0.0008
Sum $\sum$	1.160		0.372	0.1476	0.0034

$y_{cg} = \sum Ly / \sum L$

$= 0.372/1.160 = 0.321$ in.

$$\sum I'_x = \sum Ly^2 + \sum I'_1 - y_{cg}{}^2 \sum L$$

$= 0.1476 + 0.0034 - (0.321)^2 (1.160) = 0.0314 \text{ in.}^3$

4. Total Section

Element	L (in.)	y from top fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_x about own axis (in.^{3})
1	1.160	0.321	0.372	0.120	0.031
2	2.580	0.015	0.039	0.001	---
3	1.720	0.015	0.026	---	---
4	(3)(0.220) = 0.660	0.066	0.044	0.003	0.001
5	(2)(1.720) = 3.440	1.015	3.492	3.544	0.848
6	(2)(2.394) = 4.788	2.015	9.648	19.440	---
7	(2)(1.198) = 2.396	1.840	4.409	8.112	0.032
8	2.068	2.015	4.167	8.397	---
9	0.260	0.285	0.074	0.021	0.001
10	(2)(0.220) = 0.440	1.964	0.864	1.697	0.001
Sum $\sum$	19.512		23.135	41.335	0.914

$$\bar{y} = \sum Ly / \sum L$$
$$= 23.135/19.512 = 1.186 \text{ in. from top fiber}$$
$$I_x = \left[\sum Ly^2 + \sum I'_1 - \bar{y}^2 \sum L\right] t$$
$$= \left[41.335 + 0.914 - (1.186)^2 (19.512)\right] 0.030 = 0.444 \text{ in.}^4$$
$$S_{ft} = I_x / \bar{y}$$
$$= 0.444/1.186 = 0.374 \text{ in.}^3$$
$$S_{fb} = I_x / (d - \bar{y})$$
$$= 0.444/(2.030 - 1.186) = 0.526 \text{ in.}^3$$
$$A = t\sum L = (0.030)(19.512) = 0.585 \text{ in.}^2$$

3.6 Effective Section Properties

Specification provisions dealing with the strength and serviceability of flexural and compression members require the calculation of effective section properties, I_e, S_e and A_e, to account for strength and stiffness reductions due to local buckling of slender compression elements. Effective section properties are calculated by substituting a fictitious reduced width in place of the true flat width of each flat element subject to local buckling. The theoretical basis of this technique is presented in Section C2 of the Commentary and in texts such as Yu(2000)* and Hancock et. al. (2001).†

The general procedure is as follows:

1. Establish the distribution of stresses in the cross-section. The cross section stress distribution and maximum stress are given in the applicable section of Chapter C or D. Effective areas, A_e, are calculated using a uniform axial stress. Effective flexural properties, I_e and S_e, are calculated using a flexural stress distribution. Effective section properties for strength are calculated either at a maximum stress level of F_y or, in the case of members subject to flexural, flexural-torsional, torsional or lateral-torsional buckling, at the nominal buckling stress, F_n. Effective section properties for serviceability are calculated at the serviceability stress levels.
2. Under the assumed stress distribution, identify each flat element subject to either uniform compression or a stress gradient with compression on at least one edge. For each such element, determine the corresponding section from *Specification* Sections B2 through B5, which are categorized by boundary conditions of the element and stress distribution in the element. Stiffened elements are elements attached to other elements on all edges. Unstiffened elements have one edge free. There are several *Specification* sections that address complex conditions, such as flanges with stiffener lips and flat elements with multiple stiffeners. When such specialized sections apply, they should be used in lieu of the more general provisions.
3. Using the stresses and appropriate *Specification* section identified above, determine the flat width, w, stress level, f and plate buckling coefficient k of each flat compression element. These parameters are then used in *Specification* Section B2 to calculate the effective width of each element, b.
4. Using the effective widths of flat compression elements calculated above with the full properties of other elements, recalculate the section properties of the cross-section.

In many situations the calculation of section properties is iterative in nature. For example, in calculating the strength of a beam, the stresses in the cross-section are usually calculated initially using the centroid of the full cross-section. Using these stresses, the effective widths of the compression elements are determined. Using these effective widths, a new cross-section centroid is determined. Using the newly determined centroid, stresses are recalculated and the procedure is repeated until convergence, which is indicated when the centroidal axis stops shifting. Upon convergence, the effective moment of inertia, I_e, and effective section modulus, S_e are then calculated using the final effective widths.

In the examples presented herein, for beam section properties, the effective section properties are computed using one of the following two procedures:

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* Yu, W.W., *Cold-Formed Steel Design – Third Edition*, John Wiley & Sons, New York, NY, 2000.

† Hancock, G.J., Murray, T.M., Ellifritt, D.S., *Cold-Formed Steel Structures to the AISI Specification*, Marcel Dekker, Inc., New York, NY, 2001.

1. If the neutral axis of the effective section is at mid-depth of the section or closer to the tension flange than to the compression flange, the maximum stress occurs in the compression flange, thus the effective width of the compression flange and the effective width of the web elements can be calculated assuming an extreme compression fiber stress equal to the yield stress or other specified maximum stress. This case is not iterative in nature unless the web is not fully effective.
2. If the neutral axis of the effective section is closer to the compression flange than to the tension flange, the compressive stress must be known in order to calculate the effective widths of the compression elements. The compressive stresses depend upon the location of the neutral axis which in turn depends on the effective widths, thus the solution is iterative in nature. Some of the example problems demonstrate this iterative procedure.

For uniformly stressed sections, i.e. column sections, the compression stress does not vary with the distance from the centroid as in flexural members, thus iteration is not required.

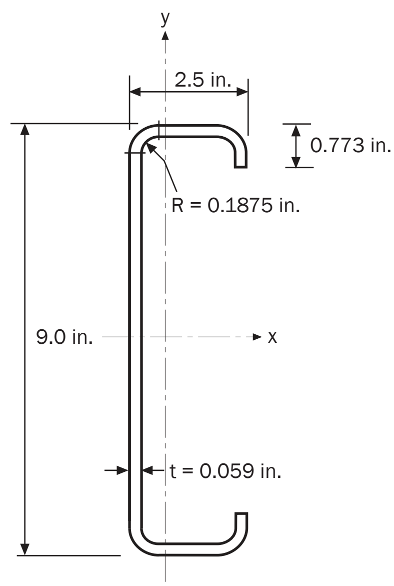
Although curved elements with large ratios of radius to thickness are also subject to local buckling, the *Specification* does not include provisions for the calculation of the effective properties of these elements. For members including such elements, the use of the Direct Strength method in Appendix 1 is recommended.

3.7 Effective Section Properties - Example Problems

The following example problems are intended to illustrate various provisions of the *Specification*, especially those involving the calculation of effective section properties. These should be used in conjunction with the other parts of the *Design Manual*. Many of the calculations are referenced in Parts II and III.

The calculations were done using the same guidelines on precision presented in Section 3.4 of Part I of the *Design Manual*.

Example I-8: C-Section With Lips - Effective Section Properties



Given:

1. Steel: F_y = 55 ksi
2. Section: 9CS2.5x059 as shown above

Required:

1. Effective section modulus, S_e, based on initiation of yielding
2. Effective area, A_e, at a uniform compressive stress of 37.25 ksi (as used in Example III-1)

Solution:

See Example I-1 for basic geometric parameters.

1. Effective section modulus, S_e, at initiation of yielding

An iterative approach is generally required.

For the first iteration, assume a compression stress of f = F_y = 55 ksi in the top fiber of the section and that the neutral axis is 4.500 in. below the top fiber.

a. Compression flange: from Section B4

$w = b = 2.007$ in.

$w/t = 2.007/0.059 = 34.02 < 60$ OK (Section B1.1(a)(1))

$$S = 1.28\sqrt{E/f} \qquad (Eq.\ B4\text{-}7)$$

$$= 1.28\sqrt{29500/55} = 29.64 \;\therefore\; w/t \geq 0.328S \;\Rightarrow \text{check effective width of flange}$$

Compute k of the flange based on stiffener lip properties.

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \leq t^4\left[115\frac{w/t}{S} + 5\right] \qquad (Eq.\ B4\text{-}8)$$

$$= 399(0.059)^4\left[\frac{34.02}{29.64} - 0.328\right]^3 \leq (0.059)^4\left[115\frac{34.02}{29.64} + 5\right]$$

$= 0.00266$ in.^{4} > 0.00166 in.^{4} $\therefore$ I_a = 0.00166 in.^{4}

d = c = 0.527 in.

θ = 90 degrees

$I_s = \left(d^3 t \sin^2\theta\right)/12$ (*Eq.* B4-10)

$= (0.527)^3(0.059)\sin^2(90°)/12 = 0.000720 \text{ in.}^4$

$R_I = I_s/I_a \le 1$ (*Eq.* B4-9)

$= 0.000720/0.00166 = 0.434 < 1$ OK

$n = \left[0.582 - \dfrac{w/t}{4S}\right] \ge \dfrac{1}{3}$ (*Eq.* B4-11)

$= \left[0.582 - \dfrac{34.02}{(4)(29.64)}\right] \ge \dfrac{1}{3}$

$= 0.295 < 1/3 \quad \therefore\ n = 1/3$

D = 0.773 in.

$D/w = 0.773/2.007 = 0.385 < 0.8$ OK (From Table B4-1)

$k = \left(4.82 - \dfrac{5D}{w}\right)(R_I)^n + 0.43 \le 4$ (From Table B4-1)

$= \left(4.82 - \dfrac{(5)(0.773)}{2.007}\right)(0.434)^{1/3} + 0.43 = 2.62 < 4$ OK

$F_{cr} = k\dfrac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\dfrac{t}{w}\right)^2$ (*Eq.* B2.1-5)

$= 2.62\dfrac{\pi^2(29500)}{12\left(1-0.3^2\right)}\left(\dfrac{1}{34.02}\right)^2 = 60.36$ ksi

$\lambda = \sqrt{\dfrac{f}{F_{cr}}}$ (*Eq.* B2.1-4)

$= \sqrt{\dfrac{55}{60.36}} = 0.955 > 0.673 \ \therefore$ flange is subject to local buckling

$\rho = (1 - 0.22/\lambda)/\lambda$ (*Eq.* B2.1-3)

$= (1 - 0.22/0.955)/0.955 = 0.806$

$b = \rho w$ (*Eq.* B2.1-2)

$= (0.806)(2.007) = 1.618$ in.

b. Stiffener lip: from Section B3.2-a

$w/t = d/t = 0.527/0.059 = 8.93$

Maximum stress in lip (by similar triangles)

$f = f_1 = 55[4.500 - 0.059/2 - 0.217]/4.500 = 51.99$ ksi

$f_2 = 55[4.500 - 0.773]/4.500 = 45.55$ ksi

$\psi = |f_2/f_1|$ (*Eq.* B3.2-1)

$= |45.55/51.99| = 0.876$

$k = \dfrac{0.578}{\psi + 0.34}$ (*Eq.* B3.2-2)

$= \dfrac{0.578}{0.876 + 0.34} = 0.475$

$$F_{cr} = k\frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{w}\right)^2 \quad (Eq.\ B2.1\text{-}5)$$

$$= 0.475\frac{\pi^2(29500)}{12\left(1-0.3^2\right)}\left(\frac{1}{8.93}\right)^2 = 158.8 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \quad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{51.99}{158.8}} = 0.572 < 0.673 \therefore \text{ lip is not subject to local buckling}$$

$$d_s' = d = 0.527 \text{ in.}$$

$$d_s = d_s'\left(R_I\right) \quad (Eq.\ B4\text{-}6)$$

$$= (0.527)(0.434) = 0.229 \text{ in.}$$

c. Web: from Section B2.3

$$w/t = 8.507/0.059 = 144.2$$

$$\psi = \left|f_2/f_1\right| \quad (Eq.\ B2.3\text{-}1)$$

Assuming the neutral axis is at the section centerline, determine the maximum flexural stress in the web by similar triangles.

$$f_1 = (55)(4.500-0.059/2-0.217)/4.500 = 51.99 \text{ ksi}$$

By symmetry

$$f_2 = -f_1 = -51.99 \text{ ksi}$$

$$\psi = \left|f_2/f_1\right| = \left|-51.99/51.99\right| = 1.0 \quad (Eq.\ B2.3\text{-}1)$$

$$k = 4+2(1+\psi)^3+2(1+\psi) \quad (Eq.\ B2.3\text{-}2)$$

$$= 4+2(1+1)^3+2(1+1) = 24.0$$

$$F_{cr} = 24.0\frac{\pi^2(29500)}{12\left(1-0.3^2\right)}\left(\frac{1}{144.2}\right)^2 = 30.77 \text{ ksi} \quad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{51.99}{30.77}} = 1.300 > 0.673 \therefore \text{ web may be subject to local buckling} \quad (Eq.\ B2.1\text{-}4)$$

$$\rho = (1-0.22/1.300)/1.300 = 0.639 \quad (Eq.\ B2.1\text{-}3)$$

$$b_e = b = (0.639)(8.507) = 5.436 \text{ in.} \quad (Eq.\ B2.1\text{-}2)$$

$$h_o/b_o = 9.000/2.500 = 3.60 < 4.0$$

$$\therefore b_1 = b_e/(3+y) \quad (Eq.\ B2.3\text{-}3)$$

$$= 5.436/(3+1) = 1.359 \text{ in.}$$

For $\psi > 0.236$

$$b_2 = b_e/2 = 5.436/2 = 2.718 \text{ in.} \quad (Eq.\ B2.3\text{-}4)$$

$$b_1 + b_2 \le w/2$$

$$1.359+2.718 = 4.077 < 8.507/2 = 4.254 \quad \therefore \text{ web is not fully effective for this iteration}$$

d. Recompute properties by parts

Represent the ineffective portion of the web as an element with a negative length

$$b_{neg} = -(4.254-4.077) = -0.177 \text{ in.}$$

Its centroidal location below the top fiber:

$y = t/2 + r + b_1 + b_{neg}/2$

$= (0.059/2) + 0.217 + 1.359 + (0.177/2) = 1.694$ in.

Element	L (in.)	y from top fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_x about own axis (in.^{3})
Top Flange	1.618	0.030	0.049	0.001	---
Bottom Flange	2.007	8.971	18.005	161.521	---
Web	8.507	4.500	38.282	172.267	51.304
Negative Web Element	-0.177	1.694	-0.300	-0.508	0.000
Top Inside Corner	0.341	0.108	0.037	0.004	0.002
Bottom Inside Corner	0.341	8.892	3.032	26.962	0.002
Top Outside Corner	0.341	0.108	0.037	0.004	0.002
Bottom Outside Corner	0.341	8.892	3.032	26.962	0.002
Top Lip	0.229	0.360	0.082	0.030	0.001
Bottom Lip	0.527	8.490	4.474	37.986	0.012
Sum $\sum$	14.075		66.730	425.229	51.325

$\bar{y} = \sum Ly / \sum L$

$= 66.730/14.075 = 4.741$ in. below top fiber

$I_x = \left[\sum I'_x + \sum Ly^2 - \bar{y}^2 \sum L\right] t$

$= \left[51.325 + 425.229 - (4.741)^2 (14.075)\right](0.059) = 9.451 \text{ in.}^4$

2. Second iteration with new neutral axis location

The calculated neutral axis location (4.741 in.) does not equal the assumed neutral axis location (4.500 in.); therefore, another iteration is required.

a. Compression flange

Since the neutral axis is below the centerline, the maximum flexural stress, F_y, will occur at the top flange. The previous solution using F_y will still be valid.

b. Stiffener lip

The change in neutral axis location will change the maximum stress and stress gradient.

$f = f_1 = (55)(4.741 - 0.059/2 - 0.217)/4.741 = 52.14 \text{ ksi}$

$f_2 = (55)(4.741 - 0.773)/4.741 = 46.03 \text{ ksi}$

$\psi = |f_2/f_1| = |46.03/52.14| = 0.883$ (*Eq.* B3.2-1)

$k = \dfrac{0.578}{0.883 + 0.34} = 0.473$ (*Eq.* B3.2-2)

$F_{cr} = 0.473 \dfrac{\pi^2 (29500)}{12(1 - 0.3^2)} \left(\dfrac{1}{8.93}\right)^2 = 158.1 \text{ ksi}$ (*Eq.* B2.1-5)

$\lambda = \sqrt{\dfrac{52.14}{158.1}} = 0.574 < 0.673 \therefore$ lip is not subject to local buckling (*Eq.* B2.1-4)

From above: $d'_s = 0.527$ in. $d_s = 0.229$ in.

c. Web

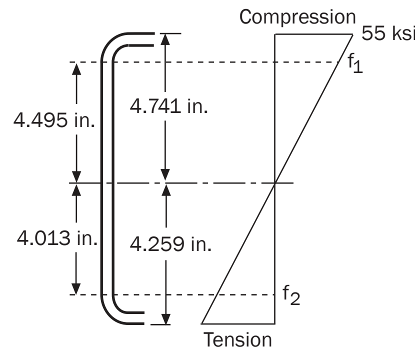
Compute new stresses at edges of web, correcting for the shift in the neutral axis.

$f_1 = 55\left(4.741 - 0.059/2 - 0.217\right)/4.741$

$= 52.14$ ksi

$f_2 = -55\left(9.000 - 4.741 - 0.059/2 - 0.217\right)/4.741$

$= -46.55$ ksi



$\psi = \left|f_2/f_1\right| = \left|-46.55/52.14\right| = 0.893$ (*Eq.* B2.3-1)

$k = 4 + 2\left(1 + 0.893\right)^3 + 2\ \left(1 + 0.893\right) = 21.35$ (*Eq.* B2.3-2)

$F_{cr} = 21.35\dfrac{\pi^2\left(29500\right)}{12\left(1 - 0.3^2\right)}\left(\dfrac{1}{144.2}\right)^2 = 27.38$ ksi (*Eq.* B2.1-5)

$\lambda = \sqrt{\dfrac{52.14}{27.38}} = 1.380 > 0.673\ \therefore$ web may be subject to local buckling (*Eq.* B2.1-4)

$\rho = \left(1 - 0.22/1.380\right)/1.380 = 0.609$ (*Eq.* B2.1-3)

$b_e = \left(0.609\right)\left(8.507\right) = 5.181$ in. (*Eq.* B2.1-2)

$b_1 = b_e/\left(3 + \psi\right)$ (*Eq.* B2.3-3)

$= 5.181/\left(3 + 0.893\right) = 1.331$ in.

For $\psi > 0.236$

$b_2 = b_e/2 = 5.181/2 = 2.591$ in. (*Eq.* B2.3-4)

$b_1 + b_2 = 1.331 + 2.591 = 3.922$ in.

Depth of compression block = 4.495 in. > 3.922 in. ; therefore, the web is not fully effective

d. Recompute properties by parts

Represent the ineffective portion of the web as an element with a negative area.

$b_{neg} = -\left(4.495 - 3.922\right) = -0.573$ in.

Its centroidal location below the top fiber

$y = t/2 + r + b_1 + b_{neg}/2$

$= \left(0.059/2\right) + 0.217 + 1.331 + \left(0.573/2\right) = 1.864$ in.

Element	L (in.)	y from top fiber (in.)	Ly ($in.^2$)	Ly^2 ($in.^3$)	I'_x about own axis ($in.^3$)
Top Flange	1.618	0.030	0.049	0.001	---
Bottom Flange	2.007	8.971	18.005	161.521	---
Web	8.507	4.500	38.282	172.267	51.304
Negative Web Element	-0.573	1.864	-1.068	-1.991	-0.016
Top Inside Corner	0.341	0.108	0.037	0.004	0.002
Bottom Inside Corner	0.341	8.892	3.032	26.962	0.002
Top Outside Corner	0.341	0.108	0.037	0.004	0.002
Bottom Outside Corner	0.341	8.892	3.032	26.962	0.002
Top Lip	0.229	0.360	0.082	0.030	0.001
Bottom Lip	0.527	8.490	4.474	37.986	0.012
Sum $\sum$	13.679		65.962	423.746	51.309

$\bar{y} = \sum Ly / \sum L$

$= 65.962/13.679 = 4.822$ in. below top fiber

$I_x = \left[\sum I'_x + \sum Ly^2 - \bar{y}^2 \sum L\right]$

$= \left[51.309 + 423.746 - (4.822)^2(13.679)\right](0.059) = 9.263$ in.^{4}

$S_e = I_x / y$

$= 9.263/4.822 = 1.92$ in.^{3}

3. Further iterations

The calculated neutral axis location (4.822 in.) does not exactly match the assumed neutral axis location (4.741 in.) but the calculated I_x and S_e are within two percent of the fully converged solution. After further iterations (not shown) the solution converges to:

$\bar{y} = 4.859$ in.

$I_x = 9.18$ in.^{4}

$S_e = 1.89$ in.^{3}

4. Effective area, A_e, at a uniform compressive stress of 37.25 ksi

a. Compression flange: taking parameters from 1 (a) above

$f = 37.25$ ksi

$S = 1.28\sqrt{29500/37.25} = 36.02$ (*Eq.* B4-7)

$w/t = 34.02 > 0.328S$ => check effective width of flange

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \quad (Eq.\ B4\text{-}8)$$

$$= 399(0.059)^4\left[\frac{34.02}{36.02} - 0.328\right]^3 \le (0.059)^4\left[115\frac{34.02}{36.02} + 5\right]$$

$= 0.00113$ in.$^4 < 0.00138$ in.^{4} $\therefore I_a = 0.00113$ in.^{4}

$R_I = I_s/I_a = 0.000720/0.00113 = 0.637$ (*Eq.* B4-9)

$$n = \left[0.582 - \frac{w/t}{4S}\right] \ge \frac{1}{3} \quad (Eq.\ B4\text{-}11)$$

$$= \left[0.582 - \frac{34.02}{(4)(36.02)}\right] \ge \frac{1}{3}$$

$= 0.346 > 1/3$ OK

$$k = \left(4.82 - \frac{5D}{w}\right)(R_I)^n + 0.43 \le 4 \quad \text{(From Table B4-1)}$$

$$k = \left(4.82 - \frac{(5)(0.773)}{2.007}\right)(0.637)^{0.346} + 0.43 = 2.906 < 4 \text{ OK}$$

$$F_{cr} = 2.906\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{34.02}\right)^2 = 66.95 \text{ ksi} \quad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{37.25}{66.95}} = 0.746 > 0.673 \therefore \text{flange is subject to local buckling} \quad (Eq.\ B2.1\text{-}4)$$

$\rho = (1 - 0.22/0.746)/0.746 = 0.945$ (*Eq.* B2.1-3)

$b = \rho w = (0.945)(2.007) = 1.897$ in. (*Eq.* B2.1-2)

b. Stiffener lip: taking parameters from 1 (b) above

$f = 37.25$ ksi

$k = 0.43$

$$F_{cr} = 0.43\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{8.93}\right)^2 = 143.8 \text{ ksi} \qquad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{37.25}{143.8}} = 0.509 < 0.673 \therefore \text{ lip is not subject to local buckling} \qquad (Eq.\ B2.1\text{-}4)$$

$d'_s = w = 0.527$ in.

$$d = d'_s(R_I) = (0.527)(0.637) = 0.336 \text{ in.} \qquad (Eq.\ B4.6)$$

c. Web: from Section B2.1

$f = 37.25$ ksi

$k = 4.0$

$$F_{cr} = 4.0\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{144.2}\right)^2 = 5.13 \text{ ksi} \qquad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{37.25}{5.13}} = 2.695 > 0.673 \therefore \text{ web is subject to local buckling} \qquad (Eq.\ B2.1\text{-}4)$$

$$\rho = (1-0.22/2.695)/2.695 \qquad (Eq.\ B2.1\text{-}3)$$

$= 0.341$

$$b = \rho w \qquad (Eq.\ B2.1\text{-}2)$$

$= (0.341)(8.507) = 2.901$ in.

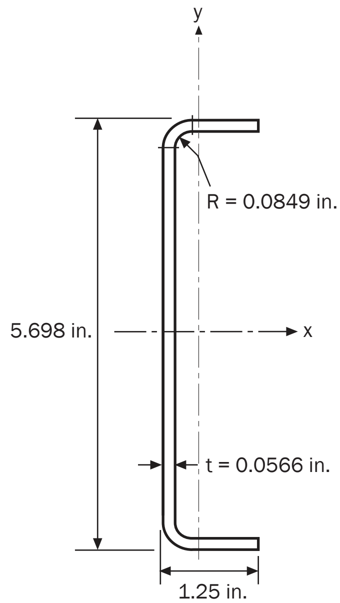
Sum of the effective widths of the elements

Element	L (in.)
Top Flange	1.897
Bottom Flange	1.897
Web	2.901
Top Inside Corner	0.341
Bottom Inside Corner	0.341
Top Outside Corner	0.341
Bottom Outside Corner	0.341
Top Lip	0.336
Bottom Lip	0.336
Sum $\sum$	8.731

$A_e = t\sum L$

$= (0.059)(8.731) = 0.515 \text{ in.}^2$

Example I-9: C-Section Without Lips - Effective Section Properties



Given:

1. Steel: F_y = 33 ksi
2. Section: SSMA Track 550T125-54 as shown above

Required:

1. Effective section modulus, S_e, at a maximum bending stress, f, of 30.93 ksi (as used in Example II-3)

Solution:

See Example I-2 for basic geometric parameters.

1. Effective section modulus, S_e, at f = 30.93 ksi

An iterative approach is generally required.

For the first iteration, assume a compression stress of f = 30.93 ksi in the top fiber of the section and a neutral axis location at the mid height of the web, 2.849 in. below the top fiber.

a. Compression flange is a uniformly compressed unstiffened element (Section B3.1)

$$w = B - r - t/2$$

$$= 1.250 - 0.113 - 0.0566/2$$

$$= 1.109 \text{ in.}$$

$$w/t = 1.109/0.0566 = 19.59 < 60 \quad \text{OK} \qquad \text{(Section B1.1(a)(3))}$$

$$k = 0.43$$

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \qquad (\textit{Eq}.\ \text{B2.1-5})$$

$$= 0.43\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{19.59}\right)^2 = 29.87 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{30.93}{29.87}} = 1.018 > 0.673 \therefore \text{ flange is subject to local buckling}$$

$$\rho = (1-0.22/\lambda)/\lambda \qquad (Eq.\ B2.1\text{-}3)$$

$$= (1-0.22/1.018)/1.018 = 0.770$$

$$b = \rho w \qquad (Eq.\ B2.1\text{-}2)$$

$$= (0.770)(1.109)$$

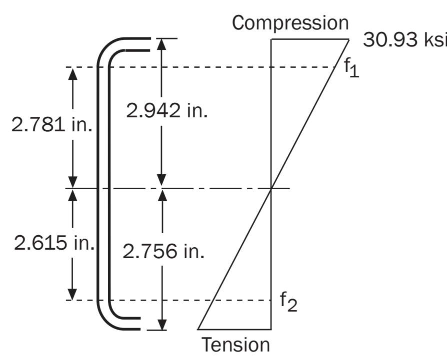
$$= 0.854 \text{ in.}$$

b. Compute new neutral axis location and check web as a stiffened element under a stress gradient.

Element	L (in.)	y from top fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_x about own axis (in.^{3})
Top Flange	0.854	0.0283	0.024	0.001	---
Top Radius	0.177	0.0693	0.012	0.001	0.000
Web	5.415	2.8490	15.427	43.952	13.232
Bottom Radius	0.177	5.6287	0.996	5.608	0.000
Bottom Flange	1.109	5.6697	6.288	35.649	----
Sum $\sum$	7.732		22.747	85.211	13.232

$$\bar{y} = \sum Ly / \sum L$$

$$= 22.747/7.732 = 2.942 \text{ in. below top fiber}$$



$$I_x = \left[\sum I'_x + \sum Ly^2 - \bar{y}^2 \sum L\right] t$$

$$= \left[13.232 + 85.211 - (2.942)^2 (7.732)\right](0.0566)$$

$$= 1.784 \text{ in.}^4$$

$$S_e = I_x/\bar{y} = 1.784/2.942 = 0.606 \text{ in.}^3$$

Check Web:

By similar triangles

$$f_1 = \left(\frac{2.942-0.113-0.0566/2}{2.942}\right)(30.93) = 29.44 \text{ ksi}$$

$$f_2 = -\left(\frac{5.698-2.942-0.113-0.0566/2}{2.942}\right)(30.93) = -27.49 \text{ ksi}$$

$$\psi = |f_2/f_1| = |-27.49/29.44| = 0.934 \qquad (Eq.\ B2.3\text{-}1)$$

$$k = 4 + 2\ (1+\psi)^3 + 2(1+\psi) \qquad (Eq.\ B2.3\text{-}2)$$

$$= 4 + 2(1+0.934)^3 + 2(1+0.934) = 22.34$$

$$w = 5.698 - 2(0.113) - 0.0566 = 5.415 \text{ in.}$$

$$w/t = 5.415/0.0566 = 95.67$$

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 22.34\frac{\pi^2 (29500)}{12(1-0.3^2)}\left(\frac{1}{95.67}\right)^2 = 65.08 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{29.44}{65.08}} = 0.673 \therefore b_e = w = 5.415 \text{ in.}$$

$h_o/b_o = 5.698/1.25 = 4.6 > 4.0$

$$\therefore b_1 = b_e/(3+\psi) \qquad (Eq.\ B2.3\text{-}6)$$

$$= 5.415/(3+0.934) = 1.376 \text{ in.}$$

$$b_2 = b_e/(1+\psi) - b_1 \qquad (Eq.\ B2.3\text{-}7)$$

$$= 5.415/(1+0.934) - 1.376 = 1.424 \text{ in.}$$

$b_1 + b_2 = 1.376 + 1.424 = 2.800$ in.

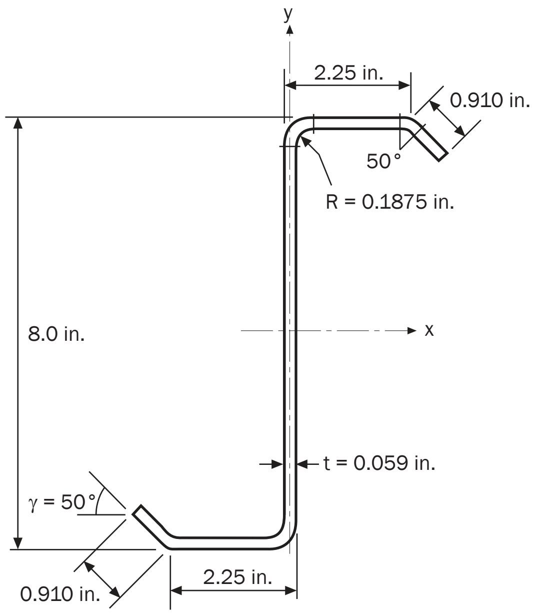
Width of compression block

$2.942 - 0.0566/2 - 0.113 = 2.801$ in. $\cong (b_1 + b_2) = 2.800$ in.

Therefore the web is fully effective and no further iteration is required.

$S_e = 0.606 \text{ in.}^3$

Example I-10: Z-Section With Lips - Effective Section Properties



Given:

1. Steel: F_y = 55 ksi
2. Section: 8ZS2.25x059 as shown above

Required:

1. Effective section modulus, S_e, based on initiation of yielding, as required in Example II-2
2. Effective moment of inertia based on procedure I of Section C3.1.1 for deflection determination at a service moment equal to 60% of the fully braced nominal moment, M_n
3. Effective area, A_e, at a uniform compressive stress of 25.9 ksi, as required in Example III-6

Solution:

See Example I-3 for basic geometric parameters.

1. Effective section modulus, S_e, at initiation of yielding

An iterative approach is generally required since the location of the neutral axis is dependant on the effective section properties.

For the first iteration, assume a compression stress of f = F_y = 55 ksi in the top fiber of the section and that the neutral axis is 4.000 in. below the top fiber.

a. Compression flange: from Section B4

$w = b = 1.889$ in.

$w/t = 1.889/0.059 = 32.0 < 60$ OK (Section B1.1(a)(1))

$$S = 1.28\sqrt{E/f} \quad (Eq.\ B4\text{-}7)$$

$$= 1.28\sqrt{29500/55} = 29.64 \ \therefore\ w/t \geq 0.328\,S \ \Rightarrow \text{check effective width of flange}$$

Compute k of flange based on stiffener lip properties.

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \leq t^4\left[115\frac{w/t}{S} + 5\right] \quad (Eq.\ B4\text{-}8)$$

$$= 399(0.059)^4\left[\frac{32.0}{29.64} - 0.328\right]^3 \leq (0.059)^4\left[115\frac{32.0}{29.64} + 5\right]$$

$= 0.00205\ \text{in.}^4 > 0.00156\ \text{in.}^4 \ \therefore\ I_a = 0.00156\ \text{in.}^4$

$d \;=\; c = 0.795$ in.

$I_s \;=\; \left(d^3 t \sin^2(\theta)\right) / 12$ (*Eq.* B4-10)

$\;=\; \left((0.795)^3 (0.059) \sin^2(50°)\right) / 12 = 0.00145$ in.^{4}

$R_I \;=\; I_s/I_a \le 1$ (*Eq.* B4-9)

$\;=\; 0.00145/0.00156 = 0.929$

$$n = \left[0.582 - \frac{w/t}{4S}\right] \ge \frac{1}{3} \qquad (Eq.\ B4\text{-}11)$$

$$= \left[0.582 - \frac{32.0}{(4)(29.64)}\right] \ge \frac{1}{3}$$

$\;=\; 0.312 < 1/3 \;\; \therefore\; n = 1/3$

$D \;=\; 0.910$ in.

$D/w = 0.910/1.889 = 0.48 < 0.8$ OK

$$k = \left(4.82 - \frac{5D}{w}\right)(R_I)^n + 0.43 \le 4 \qquad \text{(From Table B4-1)}$$

$$= \left(4.82 - \frac{(5)(0.910)}{1.889}\right)(0.929)^{1/3} + 0.43 = 2.78 < 4 \;\; \text{OK}$$

$$F_{cr} = k \frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 2.78 \frac{\pi^2 (29500)}{12\left(1-0.3^2\right)}\left(\frac{1}{32.0}\right)^2 = 72.38 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{55}{72.38}} = 0.872 > 0.673 \;\therefore\; \text{flange is subject to local buckling}$$

$\rho \;=\; (1 - 0.22/\lambda)/\lambda$ (*Eq.* B2.1-3)

$\;=\; (1 - 0.22/0.872)/0.872 = 0.857$

$b \;=\; \rho w$ (*Eq.* B2.1-2)

$\;=\; (0.857)(1.889) = 1.619$ in.

b. Stiffener lip: from Section B3.2-a

$w/t \;=\; d/t = 0.795/0.059 = 13.5$

Maximum stress in lip, f_3 (by similar triangles)

$f = f_1 = 55\left[4.000 - 0.059/2 - 0.217\left(1-\cos(50°)\right)\right]/4.000 = 53.5$ ksi

$f_2 \;=\; 55\left[4.000 - 0.059/2 - 0.217\left(1-\cos(50°)\right) - (0.795)(\sin(50°)\right]/4.000 = 45.2$ ksi

$\psi \;=\; \left|f_2/f_1\right| = \left|45.2/53.5\right| = 0.845$ (*Eq.* B3.2-1)

$$k = \frac{0.578}{\psi + 0.34} \qquad (Eq.\ B3.2\text{-}2)$$

$$= \frac{0.578}{0.845 + 0.34} = 0.488$$

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 0.488\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{13.5}\right)^2 = 71.4 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{53.5}{71.4}} = 0.866 > 0.673 \therefore \text{ lip is subject to local buckling}$$

$$\rho = (1-0.22/\lambda)/\lambda \qquad (Eq.\ B2.1\text{-}3)$$

$$= (1-0.22/0.866)/0.866 = 0.861$$

$$d'_s = b = \rho w \qquad (Eq.\ B2.1\text{-}2)$$

$$= (0.861)(0.795) = 0.684 \text{ in.}$$

$$d_s = d'_s(R_I) \qquad (Eq.\ B4\text{-}6)$$

$$= (0.684)(0.929) = 0.635 \text{ in.}$$

c. Web: from Section B2.3

$w/t = 7.507 / 0.059 = 127.2$

$$\psi = |f_2/f_1| \qquad (Eq.\ B2.3\text{-}1)$$

Assuming the neutral axis is at the mid-height of the section, find the maximum flexural stress in the web by similar triangles.

$$f_1 = (55)(4.000-0.059-0.1875)/4.000 = 51.61 \text{ ksi}$$

By symmetry

$$f_2 = -f_1 = -51.61 \text{ ksi}$$

$$\psi = |f_2/f_1| = |-51.61/51.61| = 1.0 \qquad (Eq.\ B2.3\text{-}1)$$

$$k = 4+2(1+\psi)^3+2(1+\psi) \qquad (Eq.\ B2.3\text{-}2)$$

$$= 4+2\ (1+1)^3+2(1+1) = 24.0$$

$$F_{cr} = 24.0\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{127.2}\right)^2 = 39.55 \text{ ksi} \qquad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{51.61}{39.55}} = 1.142 > 0.673 \therefore \text{ web may be subject to local buckling} \qquad (Eq.\ B2.1\text{-}4)$$

$$\rho = (1-0.22/1.142)/1.142 = 0.707 \qquad (Eq.\ B2.1\text{-}3)$$

$$b_e = b = (0.707)(7.507) = 5.307 \text{ in.} \qquad (Eq.\ B2.1\text{-}2)$$

$$h_o/b_o = 8.000/2.250 = 3.6 < 4.0$$

$$\therefore b_1 = b_e/(3+\psi) \qquad (Eq.\ B2.3\text{-}3)$$

$$= 5.307/(3+1) = 1.327 \text{ in.}$$

For $\psi > 0.236$

$$b_2 = b_e/2 = 5.307/2 = 2.654 \text{ in.} \qquad (Eq.\ B2.3\text{-}4)$$

$$b_1 + b_2 \le w/2$$

$1.327 + 2.654 = 3.981$ in. $> 7.507/2 = 3.754$ in. $\therefore$ web is fully effective for this iteration

d. Recompute properties by parts

Element	L (in.)	y from top fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_x about own axis (in.^{3})
Top Flange	1.619	0.030	0.049	0.001	---
Bottom Flange	1.889	7.971	15.057	120.021	---
Web	7.507	4.000	30.028	120.112	35.255
Top Inside Corner	0.341	0.108	0.037	0.004	0.002
Bottom Inside Corner	0.341	7.892	2.691	21.239	0.002
Top Outside Corner	0.189	0.056	0.011	0.001	0.000
Bottom Outside Corner	0.189	7.944	1.501	11.927	0.000
Top Lip	0.635	0.350	0.222	0.078	0.013
Bottom Lip	0.795	7.589	6.033	45.786	0.025
Sum $\sum$	13.505		55.629	319.169	35.297

$\bar{y} = \sum Ly/\sum L$

$= 55.629/13.505 = 4.119$ in. below top fiber

$I_x = \left[\sum I'_x + \sum Ly^2 - \bar{y}^2 \sum L\right] t$

$= \left[35.297 + 319.169 - (4.119)^2 (13.505)\right](0.059) = 7.40$ in.^{4}

Second iteration with new neutral axis location

The calculated neutral axis location (4.119 in.) does not equal the assumed neutral axis location (4.000 in.); therefore, another iteration is required.

a. Compression flange

Since the neutral axis is below the centerline, the maximum flexural stress, F_y, will occur at the top flange. The previous solution using F_y will still be valid.

b. Stiffener lip

The change in neutral axis location will change the stress gradient and consequently the maximum stress in the stiffener slightly. This may cause a minor change in the effective width of the stiffener. Neglect in this case.

c. Web

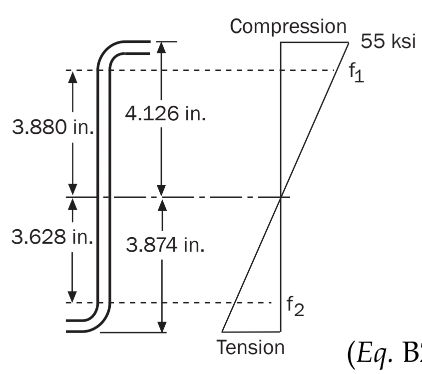
Compute new stresses at edges of web, correcting for the shift in the neutral axis.

$f_1 = 55(4.119 - 0.059 - 0.1875)/4.119$

$= 51.71$ ksi

$f_2 = -55(8.000 - 4.119 - 0.059 - 0.1875)/4.119$

$= -48.53$ ksi



$\psi = |f_2/f_1| = |-48.53/51.71| = 0.939$ (*Eq.* B2.3-1)

$k = 4 + 2(1+0.939)^3 + 2(1+0.939) = 22.46$ (*Eq.* B2.3-2)

$F_{cr} = 22.46 \dfrac{\pi^2 (29500)}{12(1-0.3^2)} \left(\dfrac{1}{127.2}\right)^2 = 37.01$ ksi (*Eq.* B2.1-5)

$\lambda = \sqrt{\dfrac{51.71}{37.01}} = 1.182 > 0.673 \therefore$ web may be subject to local buckling (*Eq.* B2.1-4)

$\rho = (1-0.22/1.182)/1.182 = 0.689$ (*Eq.* B2.1-3)

$b_e = b = (0.689)(7.507) = 5.172\text{ in.}$ (*Eq.* B2.1-2)

$b_1 = b_e/(3+\psi)$ (*Eq.* B2.3-3)

$= 5.157/(3+0.935) = 1.311\text{ in.}$

For $\psi > 0.236$

$b_2 = b_e/2 = 5.172/2 = 2.586\text{ in.}$ (*Eq.* B2.3-4)

$b_1 + b_2 = 1.313 + 2.586 = 3.899$ in.

Width of compression block

$4.119 - 0.059 - 0.1875 = 3.873\text{ in.} < 3.899\text{ in.}$ $\therefore$ web is not subject to local buckling

d. Recompute properties

There was no further reduction in the effective widths of the elements, therefore use previous solution:

$\bar{y}$ = 4.119 in. below top fiber

$I_x = 7.394\text{ in.}^4$

$S_e = I_x/\bar{y}$

$S_e = 7.394/4.119 = 1.80\text{ in.}^3$

$M_n = S_e F_y$ (*Eq.* C3.1.1-1)

$= (1.80)(55) = 99.0\text{ kip-in.}$

2. Effective moment of inertia, I_x, at a service load of 60% of M_n; M = (0.60)(99.0) = 59.4 kip-in.

A conservative approximation of flexural deflections can be obtained by performing an elastic beam analysis using the effective moment of inertia of the cross-section calculated with the maximum extreme fiber stress set to the maximum flexural stress occurring under serviceability loading. In the case of continuous beams, the average of the moments of inertia in maximum positive and negative bending can be used.

Assume the maximum compressive stress is approximately $(0.60)F_y = (0.60)(55) = 33\text{ ksi}$. The calculations are otherwise the same as above.

a. Compression flange: from Section B4

$S = 1.28\sqrt{E/f}$ (*Eq.* B4-7)

$= 1.28\sqrt{29500/33} = 38.3$

$w/t = 32.0 > 0.328\,S \Rightarrow$ check effective width of flange

Compute flange k based on stiffener lip properties.

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \quad (\textit{Eq. } \text{B4-8})$$

$$= 399(0.059)^4\left[\frac{32.0}{38.3} - 0.328\right]^3 \le 0.059^4\left[115\frac{32.0}{38.3} + 5\right]$$

$= 0.000632\text{ in.}^4 < 0.00123\text{ in.}^4$ $\therefore$ $I_a = 0.000632\text{ in.}^4$

$R_I = I_s/I_a \le 1$ (*Eq.* B4-9)

$= 0.00145/0.000632 = 2.29$ $\therefore$ $R_I = 1.0$

$$n = \left[0.582 - \frac{w/t}{4S}\right] \geq \frac{1}{3} \qquad (Eq.\ B4\text{-}11)$$

$$= \left[0.582 - \frac{32.0}{(4)(38.3)}\right] \geq \frac{1}{3}$$

$$= 0.373 > 1/3 \quad \therefore n = 0.373$$

$$k = \left(4.82 - \frac{5D}{w}\right)(R_I)^n + 0.43 \leq 4 \qquad \text{(From Table B4-1)}$$

$$= \left(4.82 - \frac{(5)(0.910)}{1.889}\right)(1.0)^{0.373} + 0.43 = 2.84 < 4 \text{ OK}$$

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 2.84\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{32.0}\right)^2 = 73.95 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{33}{73.95}} = 0.668 < 0.673 \ \therefore \text{ flange is not subject to local buckling}$$

b. Stiffener lip: from Section B3.2(a)

$w/t = d/t = 0.795/0.059 = 13.5$

Maximum stress in lip (by similar triangles)

$$f = f_1 = 33\left[4.000 - 0.059/2 - 0.217\left(1-\cos(50°)\right)\right]/4.000 = 32.12 \text{ ksi}$$

$$f_2 = 33\left[4.000 - 0.059/2 - 0.217\left(1-\cos(50°)\right) - 0.795\sin(50°)\right]/4.000 = 27.09 \text{ ksi}$$

$$\psi = |f_2/f_1| \qquad (Eq.\ B3.2\text{-}1)$$

$$= |27.09/32.12| = 0.843$$

$$k = \frac{0.578}{\psi + 0.34} \qquad (Eq.\ B3.2\text{-}2)$$

$$= \frac{0.578}{0.843 + 0.34} = 0.489$$

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$F_{cr} = 0.489\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{13.5}\right)^2 = 71.54 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{32.12}{71.54}} = 0.670 < 0.673 \ \therefore \text{ lip is not subject to local buckling}$$

$$d_s = d_s'(R_I) \qquad (Eq.\ B4\text{-}6)$$

$$= (0.795)(1.0) = 0.795 \text{ in.}$$

c. Web: from Section B2.3

By inspection, the web is fully effective at a maximum flange stress of 33 ksi, since it was shown in Parts 1 and 2 above to be fully effective at a maximum flange stress of 55 ksi.

d. Since all elements are fully effective at the assumed stress level, use the gross moment of inertia from Table I-4.

$I_x = 7.76 \text{ in.}^4$

3. Effective area, A_e, at a uniform compressive stress of 25.9 ksi

From section 2 above, it can be concluded that the flange will be fully effective at a stress of 25.9 ksi, since it is fully effective at higher stress levels.

Check lips

$k = 0.43$

$$F_{cr} = 0.43\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{13.5}\right)^2 = 62.91 \text{ ksi} \qquad (Eq.\ \text{B2.1-5})$$

$$\lambda = \sqrt{\frac{25.9}{62.91}} = 0.642 < 0.673 \therefore \text{ lips are not subject to local buckling} \qquad (Eq.\ \text{B2.1-4})$$

Check web

$k = 4.0$

$$F_{cr} = 4.0\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{127.2}\right)^2 = 6.59 \text{ ksi} \qquad (Eq.\ \text{B2.1-5})$$

$$\lambda = \sqrt{\frac{25.9}{6.59}} = 1.982 > 0.673 \therefore \text{ web is subject to local buckling} \qquad (Eq.\ \text{B2.1-4})$$

$$\rho = (1-0.22/1.982)/1.982 = 0.449 \qquad (Eq.\ \text{B2.1-3})$$

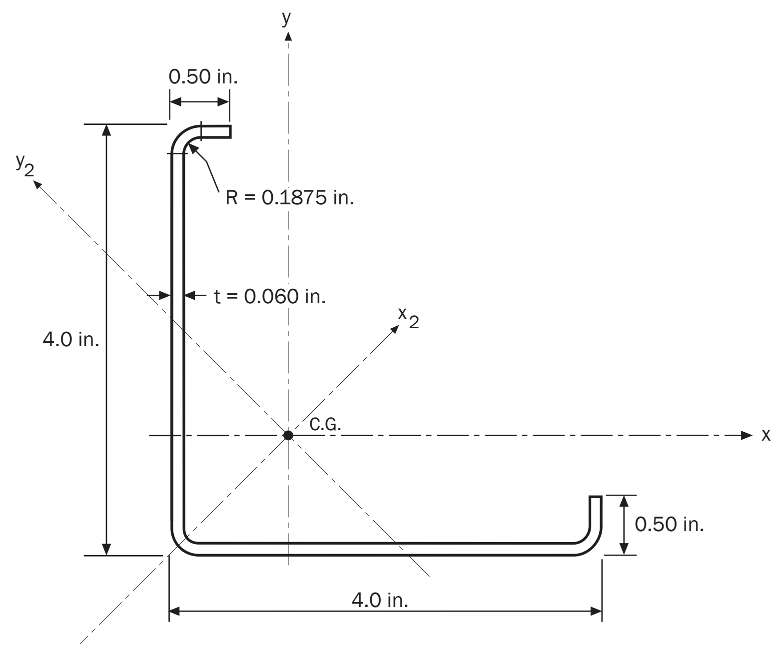
$$b_e = b = (0.449)(7.507) = 3.371 \text{ in.} \qquad (Eq.\ \text{B2.1-2})$$

To find A_e, subtract the ineffective area of the web from the gross area.

From Table I-4 or Example I-3, $A_{gross} = 0.822 \text{ in.}^2$

$$A_e = 0.822 - (7.507 - 3.371)(0.059) = 0.578 \text{ in.}^2$$

Example I-11: Equal Leg Angle With Lips - Effective Section Properties



Given:

1. Steel: F_y = 50 ksi
2. Section: 4LS4x060 as shown above

Required:

1. Effective area, A_e, at a uniform compression stress of 14.7 ksi, as required in Example III-4

Solution:

See Example I-4 for basic parameters. Treat each leg as a uniformly compressed element with an edge stiffener (Section B4).

a. Legs

w = 3.504 in.

t = 0.060 in.

$w/t = 3.504/0.060 = 58.4$

$S = 1.28\sqrt{E/f}$ (*Eq.* B4-7)

$= 1.28\sqrt{29500/14.7}$

$= 57.3$

$w/t > 0.328S$, therefore check effective width of leg

Compute k of flange based on stiffener lip properties.

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \qquad (Eq.\ B4\text{-}8)$$

$$= 399(0.060)^4\left[\frac{58.4}{57.3} - 0.328\right]^3 \le 0.060^4\left[115\frac{58.4}{57.3} + 5\right]$$

$= 0.00171 \text{ in.}^4 > 0.00158 \text{ in.}^4 \therefore I_a = 0.00158 \text{ in.}^4$

$d = c = 0.252$ in.

$I_s = \left(d^3 t \sin^2\theta\right)/12$ (*Eq.* B4-10)

$= \dfrac{(0.252)^3(0.060)\sin^2(90°)}{12} = 0.0000800 \text{ in.}^4$

$R_I = I_s/I_a \le 1$ (*Eq.* B4-9)

$= 0.0000800/0.00158 = 0.0506$

$n = \left[0.582 - \dfrac{w/t}{4S}\right] \ge \dfrac{1}{3}$ (*Eq.* B4-11)

$= \left[0.582 - \dfrac{58.4}{(4)(57.3)}\right] \ge \dfrac{1}{3}$

$= 0.327 < 1/3 \;\therefore\; n = 1/3$

$D = 0.500$ in.

$D/w = 0.500/3.504 = 0.143 < 0.25$

$k = 3.57(R_I)^n + 0.43 \le 4$ (From Table B4-1)

$= 3.57(0.0506)^{1/3} + 0.43 = 1.75$

$F_{cr} = k\dfrac{\pi^2 E}{12(1-\mu^2)}\left(\dfrac{t}{w}\right)^2$ (*Eq.* B2.1-5)

$= 1.75\dfrac{\pi^2(29500)}{12(1-0.3^2)}\left(\dfrac{1}{58.4}\right)^2 = 13.7$ ksi

$\lambda = \sqrt{\dfrac{f}{F_{cr}}}$ (*Eq.* B2.1-4)

$= \sqrt{\dfrac{14.7}{13.7}} = 1.036 > 0.673 \;\therefore$ leg is subject to local buckling

$\rho = (1-0.22/\lambda)/\lambda$ (*Eq.* B2.1-3)

$= (1-0.22/1.036)/1.036 = 0.760$

$b = \rho w$ (*Eq.* B2.1-2)

$= (0.760)(3.504) = 2.663$ in.

b. Stiffener Lips

Check Stiffener effective width

$w = d = 0.252$ in.

$w/t = 0.252/0.060 = 4.20$

$k = 0.43$

$F_{cr} = 0.43\dfrac{\pi^2(29500)}{12(1-0.3^2)}\left(\dfrac{1}{4.20}\right)^2 = 650$ ksi (*Eq.* B2.1-5)

$\lambda = \sqrt{\dfrac{14.7}{650}} = 0.150 < 0.673 \;\therefore$ stiffener is not subject to local buckling (*Eq.* B2.1-4)

$d'_s = b = w = 0.252$ in. (*Eq.* B2.1-1)

$d_s = d'_s(R_I) = (0.252)(0.0506) = 0.013$ in. (*Eq.* B4-6)

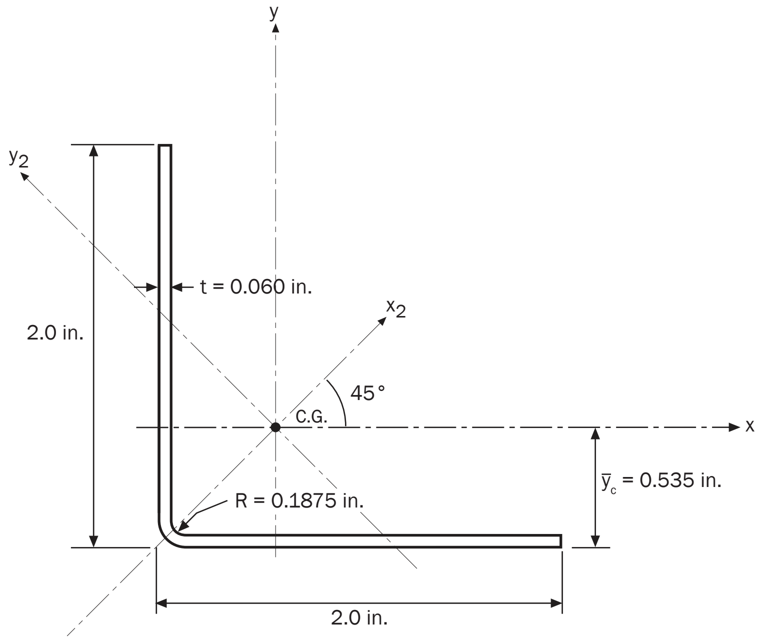
Summing the effective widths,

$$A_e = t\left[2\left(b+d_s\right)+3u\right]$$

$$= 0.060\left[2\left(2.663+0.013\right)+\left(3\right)\left(0.342\right)\right]$$

$$= 0.383 \text{ in.}^2$$

Example I-12: Equal Leg Angle Without Lips - Effective Section Properties



Given:

1. Steel: F_y = 33 ksi
2. Section: 2LU2x060 as shown above

Required:

1. Effective section modulus, S_e, at f = F_y at the extreme fibers, for flexure about the x-axis with compression on the top
2. Effective area, A_e, at f = 12.0 ksi

Solution:

Refer to Example I-5 for basic parameters.

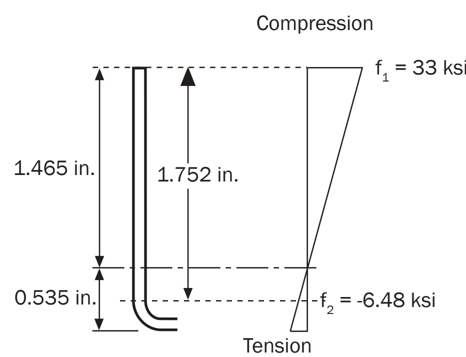
1. Effective section modulus, S_e, with compression on top (bottom flange in tension)

Treat the upstanding leg as an unstiffened element with a stress gradient (Section B3.2). Stress at tip of web will be at the yield stress in compression. At the opposite end of the web adjacent to the flange, the leg will be in tension.

$$w = 2.00 - 0.060 - 0.1875 = 1.752 \text{ in.}$$

$$f_1 = 33 \text{ ksi}$$

$$f_2 = 33.0\frac{-(0.535 - 0.060 - 0.1875)}{2.000 - 0.535} = -6.48 \text{ ksi}$$



$$\psi = |f_2/f_1| \qquad (Eq.\ B3.2\text{-}1)$$

$$= |-6.48/33| = 0.196$$

$$k = 0.57 + 0.21\psi + 0.07\psi^2 \qquad (Eq.\ B3.2\text{-}5)$$

$$= 0.57 + 0.21(0.196) + 0.07(0.196)^2 = 0.614$$

$$F_{cr} = k\frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$F_{cr} = 0.614\frac{\pi^2(29500)}{12\left(1-0.3^2\right)}\left(\frac{0.060}{1.752}\right)^2 = 19.2\text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{33}{19.2}} = 1.311 > 0.673(1+\psi) = 0.805 \therefore \text{ leg is subject to local buckling}$$

$$\rho = (1+\psi)\frac{\left(1-\frac{0.22(1+\psi)}{\lambda}\right)}{\lambda} \qquad (Eq.\ B3.2\text{-}4)$$

$$= (1+0.196)\frac{\left(1-\frac{0.22(1+0.196)}{1.311}\right)}{1.311} = 0.729$$

$$b = \rho w \qquad (Eq.\ B2.1\text{-}2)$$

$$= (0.729)(1.752) = 1.277\text{ in.}$$

Effective section properties

Element	L (in.)	y from top fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I′$_x$ about own axis (in.^{3})
web	1.277	1.114	1.423	1.585	0.174
corner	0.342	1.891	0.647	1.223	0.002
flange	1.752	1.970	3.451	6.799	---
Sum $\sum$	3.371		5.521	9.607	0.176

$$\bar{y} = \sum Ly/\sum L = 5.521/3.371 = 1.638\text{ in. below top fiber}$$

The neutral axis has shifted, which will result in a change in the stress gradient in the upstanding leg of the angle. Recompute the effective properties with the new neutral axis position.

$$f_1 = 33\text{ ksi}$$

$$f_2 = 33.0\frac{-(2.00-1.638-0.060-0.1875)}{1.638} = -2.31\text{ ksi}$$

$$\psi = \left|-2.31/33\right| = 0.0700 \qquad (Eq.\ B3.2\text{-}1)$$

$$k = 0.57+0.21(0.0700)+0.07(0.0700)^2 = 0.585 \qquad (Eq.\ B3.2\text{-}5)$$

$$F_{cr} = 0.585\frac{\pi^2(29500)}{12\left(1-0.3^2\right)}\left(\frac{0.060}{1.752}\right)^2 = 18.3\text{ ksi} \qquad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{33}{18.3}} = 1.343 \qquad (Eq.\ B2.1\text{-}4)$$

$$1.343 > 0.673(1+\psi) = 0.720 \therefore \text{ leg is subject to local buckling}$$

$$\rho = (1+0.0700)\frac{\left(1-\frac{0.22(1+0.0700)}{1.343}\right)}{1.343} = 0.657 \qquad (Eq.\ B3.2\text{-}4)$$

$$b = (0.657)(1.752) = 1.151 \text{ in.} \qquad (Eq.\ B2.1\text{-}2)$$

Effective section properties

Element	L (in.)	y from top fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I′$_x$ about own axis (in.^{3})
web	1.151	1.177	1.355	1.595	0.127
corner	0.342	1.891	0.647	1.223	0.002
flange	1.752	1.970	3.451	6.799	----
Sum $\sum$	3.245		5.453	9.617	0.129

$$\bar{y} = \sum Ly/\sum L = 5.453/3.245 = 1.680 \text{ in. below top fiber}$$

The neutral axis has shifted again, which will result in a change in the stress gradient in the up-standing leg of the angle. Recompute the effective properties with the new neutral axis position.

$$f_1 = 33 \text{ ksi}$$

$$f_2 = 33.0\frac{-(2.00-1.680-0.060-0.1875)}{1.680} = -1.42 \text{ ksi}$$

$$\psi = \left|-1.42/33\right| = 0.0430 \qquad (Eq.\ B3.2\text{-}1)$$

$$k = 0.57+0.21(0.0430)+0.07(0.0430)^2 = 0.579 \qquad (Eq.\ B3.2\text{-}5)$$

$$F_{cr} = 0.579\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.060}{1.752}\right)^2 = 18.1 \text{ ksi} \qquad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{33}{18.1}} = 1.350 \qquad (Eq.\ B2.1\text{-}4)$$

$1.350 > 0.673(1+\psi)$ ∴ leg is subject to local buckling

$$\rho = (1+0.0430)\frac{\left(1-\frac{0.22(1+0.0430)}{1.350}\right)}{1.350} = 0.641 \qquad (Eq.\ B3.2\text{-}4)$$

$$b = (0.641)(1.752) = 1.123 \text{ in.} \qquad (Eq.\ B2.1\text{-}2)$$

Effective section properties

Element	L (in.)	y from top fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I′$_x$ about own axis (in.^{3})
web	1.123	1.191	1.337	1.593	0.118
corner	0.342	1.891	0.647	1.223	0.002
flange	1.752	1.970	3.451	6.799	----
Sum $\sum$	3.217		5.435	9.615	0.120

$$\bar{y} = \sum Ly/\sum L = 5.435/3.217 = 1.689 \text{ in. below top fiber}$$

The neutral axis has shifted again, but the change is less than 0.01 in. Compute the effective properties with the new neutral axis position.

$$I_x = \left[\sum I_x' + \sum Ly^2 - \bar{y}^2\sum L\right]t$$

$$= \left[0.120+9.615-(1.689)^2(3.217)\right]0.060 = 0.0335 \text{ in.}^4$$

$S_t = \dfrac{I_x}{y_t} = \dfrac{0.0335}{1.689} = 0.0198\text{ in.}^3$

$S_b = \dfrac{I_x}{y_b} = \dfrac{0.0335}{2.000 - 1.689} = 0.108\text{ in.}^3$

2. Effective area, A_e, at f = 12.0 ksi

Treat flanges as uniformly compressed unstiffened element (Section B3.1)

f = 12.0 ksi

k = 0.43

w = 1.752 in.

$$F_{cr} = k\frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 0.43\frac{\pi^2\left(29500\right)}{12\left(1-0.3^2\right)}\left(\frac{0.060}{1.752}\right)^2 = 13.45\text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{12.0}{13.45}} = 0.945 > 0.673 \therefore \text{ leg is subject to local buckling}$$

$$\rho = \left(1 - 0.22/\lambda\right)/\lambda \qquad (Eq.\ B2.1\text{-}3)$$

$$= \left(1 - 0.22/0.945\right)/0.945 = 0.812$$

$$b = \rho w = \left(0.812\right)\left(1.752\right) = 1.423\text{ in.} \qquad (Eq.\ B2.1\text{-}2)$$

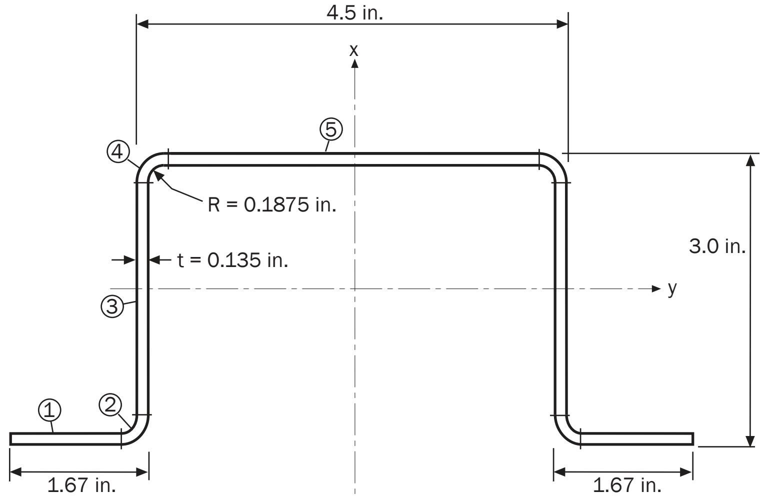
Effective area

$$A_e = t\sum L$$

$$= \left(0.060\right)\left[\left(2\right)\left(1.423\right) + 0.342\right]$$

$$= 0.191\text{ in.}^2$$

Example I-13: Hat Section - Effective Section Properties Using Inelastic Reserve Capacity



Given:

1. Steel: F_y = 50 ksi
2. Section: 3HU4.5x135 as shown in sketch above
3. Top flange continuously braced

Required:

1. Determine the nominal flexural strength, M_{ny}, with the top flange in compression, based on initiation of yielding.
2. Determine the nominal flexural strength, M_{ny}, with the top flange in compression, based on inelastic reserve capacity.
3. Determine the effective area, A_e, at a uniform compressive stress of 50 ksi.

Solution:

Refer to Example I-6 for derivation of basic parameters.

1. Nominal flexural strength based on initiation of yielding (Section C3.1.1.a)

Computation of I_y, first approximation:

Assume a compressive stress of f = F_y = 50 ksi in the top fiber of the section.

Assume the web is fully effective.

Element 3:

$h/t = 2.355/0.135 = 17.44 < 200$ OK (Section B1.2-(a)). Assumed fully effective

Element 5:

$w/t = 3.855/0.135 = 28.56 < 500$ OK (Section B1.1-(a)-(2))

k = 4.0 (fully stiffened element)

$$F_{cr} = k\frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 4\frac{\pi^2(29500)}{12\left(1-0.3^2\right)}\left(\frac{1}{28.56}\right)^2 = 130.8 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{50}{130.8}} = 0.618 < 0.673 \ \therefore \text{ flange is not subject to local buckling}$$

$$b = w \qquad (Eq.\ B2.1\text{-}1)$$

$$= 3.855 \text{ in.}$$

Effective section properties about y-axis:

Element	L (in.)	x from top fiber (in.)	Lx (in.^{2})	Lx^2 (in.^{3})	I'_y about own axis (in.^{3})
1	(2)(1.348) = 2.696	2.932	7.905	23.176	---
2	(2)(0.401) = 0.802	2.840	2.278	6.469	0.004
3	(2)(2.355) = 4.710	1.500	7.065	10.598	2.177
4	(2)(0.401) = 0.802	0.161	0.129	0.021	0.004
5	3.855	0.068	0.262	0.018	---
Sum $\sum$	12.865		17.639	40.282	2.185

Distance of neutral axis from top fiber,

$$\bar{x} = \sum Lx / \sum L = 17.639/12.865 = 1.371 \text{ in.}$$

$$I_y = \left[\sum Lx^2 + \sum I'_y - \bar{x}^2 \sum L\right] t$$

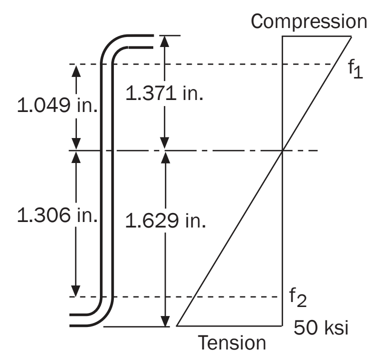
$$= \left[40.282 + 2.185 - (1.371)^2 (12.865)\right](0.135)$$

$$= 2.469 \text{ in.}^4$$

Since the distance of the top compression fiber from the neutral axis is less than one half of the beam depth , a compressive stress of f equal to F_y at the top fiber will not govern as assumed at the beginning of item 1. The actual compressive stress will be less than F_y, so the flange will still be fully effective. The tension flange will yield first.

Therefore,

Check web (element 3) under new assumed stress distribution



$$f_1 = (1.049/1.629)(50) = 32.20 \text{ ksi (compression)}$$

$$f_2 = -(1.306/1.629)(50) = -40.09 \text{ ksi (tension)}$$

$$\psi = |f_2/f_1| = |-40.09/32.20| = 1.245 \qquad (Eq.\ B2.3\text{-}1)$$

$$k = 4 + 2(1+\psi)^3 + 2(1+\psi) \qquad (Eq.\ B2.3\text{-}2)$$

$$= 4 + 2(1+1.245)^3 + 2(1+1.245)$$

$$k = 31.12$$

$$F_{cr} = 31.12 \frac{\pi^2 (29500)}{12(1-0.3^2)} \left(\frac{1}{17.44}\right)^2 = 2728 \text{ ksi} \qquad (Eq.\ B2.1\text{-}5)$$

$\lambda = \sqrt{\frac{f}{F_{cr}}}$ (*Eq.* B2.1-4)

$= \sqrt{\frac{32.20}{2728}} = 0.109 < 0.673$

$b = w$ (*Eq.* B2.1-1)

$b_e = 2.355$ in.

$h_o/b_o = 3.0/4.5 = 0.67 < 4.0$

$b_1 = b_e/(3+\psi)$ (*Eq.* B2.3-3)

$= 2.355/(3+1.245) = 0.555$ in.

For $\psi > 0.236$

$b_2 = b_e/2$ (*Eq.* B2.3-4)

$= 2.355/2 = 1.178$ in.

$b_1+b_2 = 0.555 + 1.178 = 1.733$ in. > 1.049 in. (compression portion of web)

Therefore, web is fully effective.

$S_e = I_y/(d-\bar{x}) = 2.469/(3.000-1.371) = 1.516 \text{ in.}^3$

$M_n = S_eF_y$ (*Eq.* C3.1.1-1)

$= (1.516)(50)$

$= 75.8$ kip-in.

2. Nominal flexural strength based on inelastic reserve capacity (Section C3.1.1.b)

Compute the maximum compression strain.

$\lambda_1 = \frac{1.11}{\sqrt{F_y / E}}$ (*Eq.* C3.1.1-3)

$= \frac{1.11}{\sqrt{50 / 29500}} = 26.96$

$\lambda_2 = \frac{1.28}{\sqrt{F_y/E}}$ (*Eq.* C3.1.1-4)

$= \frac{1.28}{\sqrt{50/29500}} = 31.09$

w/t of compression flange = 28.56

For $26.96 = \lambda_1 < w/t < \lambda_2 = 31.09$

$C_y = 3-2\left(\frac{w/t-\lambda_1}{\lambda_2-\lambda_1}\right)$ (*Eq.* C3.1.1-2)

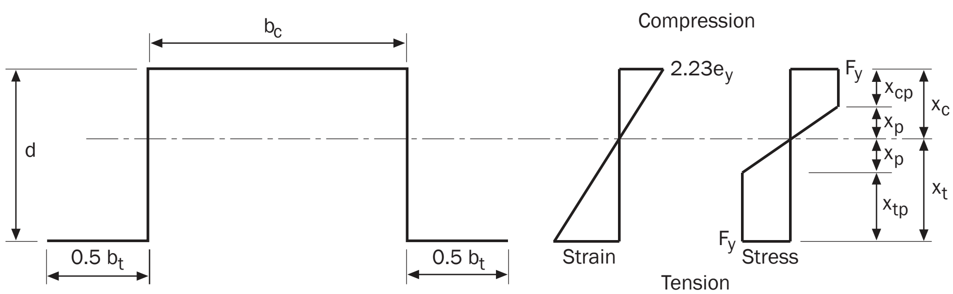
$= 3-2\left(\frac{28.56-26.96}{31.09-26.96}\right) = 2.23$

Therefore, the maximum compression strain is 2.23 times the yield strain, e_y. The tension strain is not limited.

Compute location of e_y on a strain diagram such that the maximum compression strain does not exceed 2.23 e_y and the summation of longitudinal forces is zero. Using the equations from Reck, Pekoz and Winter, "Inelastic Strength of Cold-Formed Steel Beams", Journal of the Structural Division, November 1975, ASCE:

Approximate distance from neutral axis to the outer compression fiber, y_c (not considering the effect of radii):

t = 0.135 in.
b_t = $2(1.670) = 3.340$ in.
b_c = 4.500 in.
d = 3.000 in.
x_c = $(1/4)(b_t - b_c + 2d)$
= $(1/4)[3.340 - 4.500 + 2(3.000)] = 1.210$ in.
x_p = x_c/C_y
= 1.210/2.23 = 0.543 in.
x_t = $d - x_c$
= $3.000 - 1.210 = 1.790$ in.
x_{cp} = $x_c - x_p$
x_{cp} = $1.210 - 0.543 = 0.667$ in.
x_{tp} = $x_t - x_p = 1.790 - 0.543 = 1.247$ in.



Summing moments of stresses in component plates:

$$M_n = F_y t\left\{b_c x_c + 2x_{cp}\left[x_p + \frac{x_{cp}}{2}\right] + \frac{4}{3}x_p^{\,2} + 2x_{tp}\left[x_p + \frac{x_{tp}}{2}\right] + b_t x_t\right\}$$

$$M_n = (50.0)(0.135)\left\{\begin{array}{l}(4.500)(1.210) + (2)(0.667)\left[0.543 + \dfrac{0.667}{2}\right] + \dfrac{4}{3}(0.543)^2 \\ \qquad +2(1.247)\left[0.543 + \dfrac{1.247}{2}\right] + (3.340)(1.790)\end{array}\right\}$$

M_n = 107.3 kip-in.

M_n shall not exceed $1.25S_eF_y = 1.25(75.8) = 94.8$ kip-in. CONTROLS

Therefore

$M_n = 1.25S_eF_y = 94.8$ kip-in.

The inelastic reserve capacity can be used assuming the following conditions are met:

(1) The member is not subject to twisting, lateral, torsional, or flexural-torsional buckling.

(2) The effect of cold-forming is not included in determining the yield stress, F_y.

(3) The ratio of depth of the compressed portion of the web to its thickness does not exceed λ_1:

$(x_c - r - t/2)/t = (1.210 - 0.255 - 0.135/2)/0.135 = 6.6 < \lambda_1 = 26.96$ OK

(4) The shear force does not exceed $0.35F_y$ times the web area, ht, for ASD, and $0.6F_y$ht for LRFD.

(5) The angle between any web and the vertical does not exceed 30°.

3. Effective area, A_e, at a uniform compressive stress of f=50 ksi (Section C4)

Element 5: Uniformly Compressed Stiffened Element (Section B2.1)

$w/t = 3.855/0.135$

$= 28.56$

$k = 4.0$

$$F_{cr} = 4.0\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{28.56}\right)^2 = 130.8 \text{ ksi} \quad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{50}{130.8}} = 0.618 < 0.673 \text{ (flange is fully effective)} \quad (Eq.\ B2.1\text{-}4)$$

$b = w$ (*Eq.* B2.1-1)

$= 3.855$ in.

Element 3: Uniformly Compressed Element with a Simple Lip Edge Stiffener (Section B4)

$w = 2.355$ in.

$w/t = 2.355/0.135 = 17.4 < 60$ OK (Section B1.1(a)(1))

$S = 1.28\sqrt{E/f}$ (*Eq.* B4-7)

$= 1.28\sqrt{29500/50} = 31.1 \ \therefore\ w/t \ge 0.328S \ \Rightarrow$ check effective width of element

Compute k of element based on stiffener lip (element 1) properties.

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \quad (Eq.\ B4\text{-}8)$$

$$= 399(0.135)^4\left[\frac{17.4}{31.1} - 0.328\right]^3 \le (0.135)^4\left[115\frac{17.4}{31.1} + 5\right]$$

$= 0.00164 \text{ in.}^4 < 0.0230 \text{ in.}^4 \ \therefore\ I_a = 0.00164 \text{ in.}^4$

$d = 1.348$ in.

$\theta = 90$ degrees

$I_s = (d^3 t \sin^2\theta)/12$ (*Eq.* B4-10)

$= (1.348)^3(0.135)\sin^2(90°)/12 = 0.0276 \text{ in.}^4$

$R_I = I_s/I_a \le 1$ (*Eq.* B4-9)

$= 0.0276/0.00164 = 16.8 > 1 \ \therefore\ R_I = 1$

$D = 1.67$ in.

$D/w = 1.67/2.355 = 0.71 < 0.8$ OK (From Table B4-1)

$$k = \left(4.82 - \frac{5D}{w}\right)(R_I)^n + 0.43 \le 4 \quad \text{(From Table B4-1)}$$

$$= \left(4.82 - \frac{(5)(1.67)}{2.355}\right)(1)^n + 0.43 = 1.70 < 4 \text{ OK}$$

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \quad (Eq.\ B2.1\text{-}5)$$

$$= 1.70\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{17.4}\right)^2 = 150 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \quad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{50}{150}} = 0.577 < 0.673 \therefore \text{ element is not subject to local buckling}$$

$$b = w$$

$$= 2.355 \text{ in.}$$

Element 1: Uniformly Compressed Unstiffened Element (Section B3.1 and B4)

$$w/t = 1.348/0.135 = 9.99$$

$$k = 0.43$$

$$F_{cr} = 0.43\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{1}{9.99}\right)^2 = 115 \text{ ksi} \quad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \quad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{50}{115}} = 0.659 < 0.673 \therefore \text{ element is not subject to local buckling}$$

$$d'_s = b = w \quad (Eq.\ B2.1\text{-}1)$$

$$= 1.348 \text{ in.}$$

$$d_s = d'_s(R_I) \quad (Eq.\ B4\text{-}6)$$

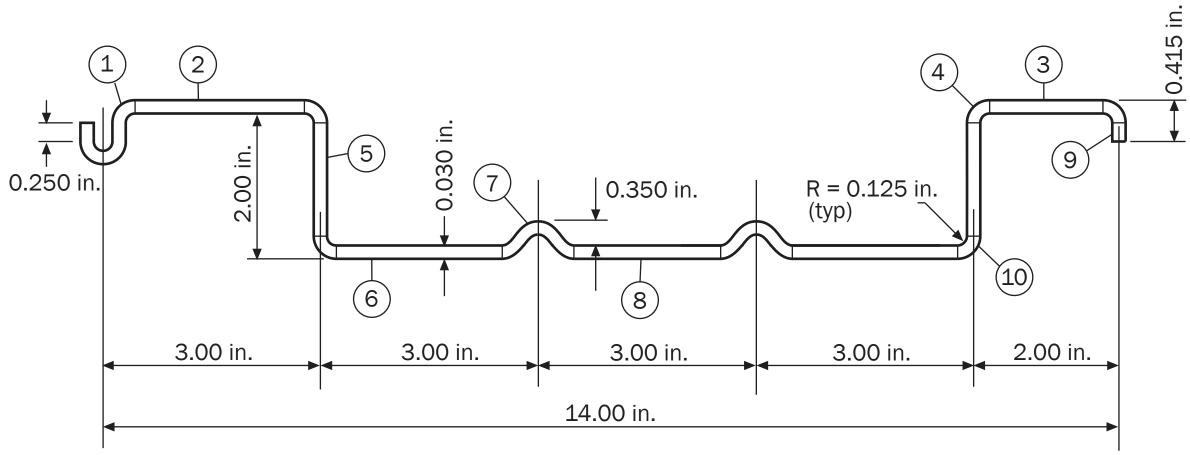
$$= (1.348)(1.0) = 1.348 \text{ in.}$$

Effective Area = Gross Area:

Element	L (in.)
1	(2)(1.348) = 2.696
2	(2)(0.401) = 0.802
3	(2)(2.355) = 4.710
4	(2)(0.401) = 0.802
5	3.855
Sum $\sum$	12.865

$$A_e = t\sum L = (0.135)(12.865) = 1.74 \text{ in.}^2$$

Example I-14: Wall Panel Section with Intermediate Stiffeners - Effective Section Properties



Given:

1. Steel: F_y = 50 ksi
2. Section: Shown in sketch above. Refer to Example I-7 for gross properties of elements. Section is assumed to be fully braced against member buckling.

Required:

1. Nominal flexural strength per panel, M_n, for positive and negative bending.
2. Effective moment of inertia, I_{eff}, at a moment of $0.6M_n$ with compression on the top. Use Procedure II from Part (b) of Section B2.1 to compute effective widths of stiffened elements at service load.

Solution:

1. Section modulus, S_e, for nominal flexural strength - compression on top

From Example I-7, the neutral axis is 1.186 in. below the top fibers of the gross cross-section (below the mid depth); therefore, the compression stress at the top fiber will govern and will equal F_y.

Unstiffened Lip of Element 1 from Section B3.2(a)

Although a more precise check of this unstiffened element with a stress gradient could be conducted using Section B3.2, since this is a very short element perform a simpler, conservative, preliminary check using Section B3.1 and $f=F_y$.

w = 0.250 in.
k = 0.43
f = F_y (Conservative assumption for preliminary check)

$$F_{cr} = k\frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 0.43\frac{\pi^2\left(29500\right)}{12\left(1-0.3^2\right)}\left(\frac{0.030}{0.250}\right)^2 = 165.1 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{50}{165.1}} = 0.550 < 0.673 \ \therefore \text{ lip is not subject to local buckling}$$

No need to calculate more precisely, since the lip is fully effective using conservative assumptions

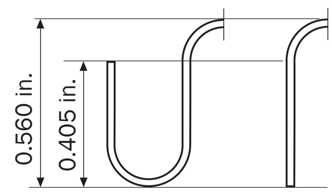
$b = w = 0.250$ in.

Stiffened Flat Element of Element 1 with Stress Gradient from Section B2.3

By inspection, the stress levels will be lower than those used to check the unstiffened element and k will be at least 4.0. Consequently, the element will be fully effective, since the unstiffened element is fully effective and the length is the same.

Element 2 from Section B4(a)

Evaluate element 2 as a uniformly compressed element with an edge stiffener. Prior editions of the *Specification* included provisions for evaluating the effective width of a uniformly compressed element with stiffeners other than simple lips; however, these provisions are no longer included. For purposes of evaluating the plate buckling coefficient, k, of the flange, the complex stiffener (element 1) will be simplified in a conservative way. A simple edge stiffener extending to the bottom of the complex stiffener with a flat width of 0.405 in. will be substituted, neglecting the beneficial contribution of the rest of the stiffener. For a more exact solution, the use of the Direct Strength provisions in *Specification* Appendix 1 is recommended.



$w = 3.000 - 3(0.125 + 0.030/2) = 2.580$ in.

$f = F_y = 50$ ksi

$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/50} = 31.09$ (*Eq.* B4-7)

$w/t = 2.580/0.030 = 86.0 > 0.328S \therefore$ check effective width

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \quad (Eq.\ B4\text{-}8)$$

$$= 399(0.030)^4\left[\frac{86.0}{31.09} - 0.328\right]^3 \le (0.030)^4\left[115\frac{86.0}{31.09} + 5\right]$$

$= 0.00468\text{ in.}^4 > 0.000262\text{ in.}^4 \therefore I_a = 0.000262\text{ in.}^4$

Using the simplified stiffener lip

$$I_s = \left(d^3 t \sin^2\theta\right)/12 \quad (Eq.\ B4\text{-}10)$$

$$= (0.405)^3(0.030)\sin^2(90°)/12 = 0.000166\text{ in.}^4$$

$$R_I = I_s/I_a \le 1 \quad (Eq.\ B4\text{-}9)$$

$$= 0.000166/0.000262 = 0.634 < 1.0 \quad \therefore R_I = 0.634$$

$$n = \left(0.582 - \frac{w/t}{4S}\right) \ge \frac{1}{3} \quad (Eq.\ B4\text{-}11)$$

$$= \left(0.582 - \frac{86.0}{4(31.09)}\right) \ge \frac{1}{3}$$

$$= -0.110 < 1/3 \therefore n = 1/3$$

Using the simplified stiffener lip flat length of 0.405 in.,

$D/w = 0.560/2.580 = 0.217 < 0.25$; therefore,

$$k = 3.57(R_I)^n + 0.43 \le 4 \quad \text{(From Table B4-1)}$$

$$= 3.57(0.634)^{1/3} + 0.43 = 3.50$$

$$F_{cr} = k\frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 3.50\frac{\pi^2\left(29500\right)}{12\left(1-0.3^2\right)}\left(\frac{0.030}{2.580}\right)^2 = 12.62\text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{50.0}{12.62}} = 1.99 > 0.673 \therefore \text{ element is subject to local buckling}$$

$$\rho = \left(1-0.22/\lambda\right)/\lambda \qquad (Eq.\ B2.1\text{-}3)$$

$$= \left(1-0.22/1.99\right)/1.99 = 0.447$$

$$b = \rho w \qquad (Eq.\ B2.1\text{-}2)$$

$$= \left(0.447\right)\left(2.580\right) = 1.153\text{ in.}$$

Element 9 from Section B3.2(a)

Although a more precise check of this unstiffened element with a stress gradient could be conducted using Section B3.2, since this is a very short element, perform a simpler conservative preliminary check using Section B3.1 and $f=F_y$.

$$w = 0.415-0.030-0.125 = 0.260\text{ in.}$$

$$k = 0.43$$

$f < F_y$, but use F_y as a conservative value

$$F_{cr} = 0.43\frac{\pi^2\left(29500\right)}{12\left(1-0.3^2\right)}\left(\frac{0.030}{0.260}\right)^2 = 152.6\text{ ksi} \qquad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}}$$

$$= \sqrt{\frac{50.0}{152.6}} = 0.572 < 0.673 \therefore \text{ element is not subject to local buckling} \qquad (Eq.\ B2.1\text{-}4)$$

$$d'_s = b = w = 0.260\text{ in.} \qquad (Eq.\ B2.1\text{-}1)$$

Element 3 from Section B4(a)

$$w = 2.000-2\left(0.125+0.030/2\right) = 1.720\text{ in.}$$

$$f = F_y = 50\text{ ksi}$$

$$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/50} = 31.09 \qquad (Eq.\ B4\text{-}7)$$

$$w/t = 1.720/0.030 = 57.33 > 0.328S \therefore \text{ check effective width}$$

$$I_a = 399t^4\left[\frac{w/t}{S}-0.328\right]^3 \le t^4\left[115\frac{w/t}{S}+5\right] \qquad (Eq.\ B4\text{-}8)$$

$$= 399\left(0.030\right)^4\left[\frac{57.33}{31.09}-0.328\right]^3 \le \left(0.030\right)^4\left[115\frac{57.33}{31.09}+5\right]$$

$$= 0.00113\text{ in.}^4 > 0.000176\text{ in.}^4 \therefore I_a = 0.000176\text{ in.}^4$$

$$I_s = (d^3 t\sin^2\theta)/12 \qquad (Eq.\ B4\text{-}10)$$

$$= \left(0.260\right)^3\left(0.030\right)\sin^2\left(90°\right)/12 = 0.0000439\text{ in.}^4$$

$R_I = I_s/I_a \le 1$ (*Eq.* B4-9)

$= 0.0000439/0.000176 = 0.249$

$n = \left(0.582 - \frac{w/t}{4S}\right) \ge \frac{1}{3}$ (*Eq.* B4-11)

$= \left(0.582 - \frac{57.33}{4(31.09)}\right) \ge \frac{1}{3}$

$= 0.121 < 1/3 \quad \therefore\ n = 1/3$

$D/w = 0.415/1.720 = 0.241 < 0.25$

$k = 3.57(0.249)^{1/3} + 0.43 = 2.68$ (From Table B4.2)

$F_{cr} = 2.68\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.030}{1.720}\right)^2 = 21.74$ ksi (*Eq.* B2.1-5)

$\lambda = \sqrt{\frac{f}{F_{cr}}} = \sqrt{\frac{50.0}{21.74}} = 1.517 > 0.673\ \therefore$ element is subject to local buckling (*Eq.* B2.1-4)

$\rho = (1-0.22/\lambda)/\lambda$ (*Eq.* B2.1-3)

$= (1-0.22/1.517)/1.517 = 0.564$

$b = \rho w$ (*Eq.* B2.1-2)

$= (0.564)(1.720) = 0.970$ in.

Effective width of element 9

$d_s = d_s'(R_I) = (0.260)(0.249) = 0.065$ in. (*Eq.* B4-6)

Effective section properties about x-axis, assuming element 5 is fully effective:

Element	L (in.)	y from top fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_x about own axis (in.^{3})
1	1.160	0.321	0.372	0.120	0.031
2	1.153	0.015	0.017	---	---
3	0.970	0.015	0.015	---	---
4	0.660	0.066	0.044	0.003	0.001
5	3.440	1.015	3.492	3.544	0.848
6	4.788	2.015	9.648	19.440	---
7	2.396	1.840	4.409	8.112	0.032
8	2.068	2.015	4.167	8.397	---
9	0.065	0.188	0.012	0.002	---
10	0.440	1.964	0.864	1.697	0.001
Sum $\sum$	17.140		23.040	41.315	0.913

$\bar{y} = \sum Ly/\sum L = 23.040/17.140 = 1.344$ in.; below mid depth as assumed

$I_x = \left[\sum Ly^2 + \sum I'_x - \bar{y}^2\sum L\right]t$

$= \left[41.315 + 0.913 - (1.344)^2(17.140)\right](0.030) = 0.338$ in.^{4}

$S_{xt} = I_x/y_{cg} = 0.338/1.344 = 0.251$ in.^{3}

$M_n = S_eF_y = 0.251(50) = 12.6$ kip-in (*Eq.* C3.1.1-1)

Element 5 from Section B2.3(a): check assumption that element is fully effective

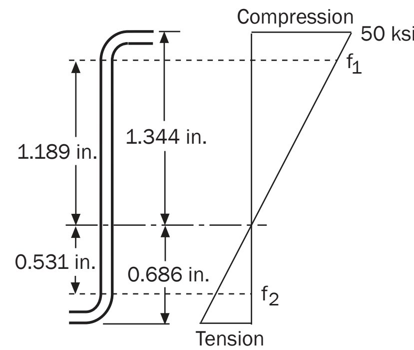
y_{cg} = 1.344 in.

$f_1 = \left[(1.344 - 0.125 - 0.030)/1.344\right](50)$

= 44.23 ksi

$f_2 = -\left[(2.030 - 0.125 - 0.030 - 1.344)/1.344\right](50)$

= –19.75 ksi



$\psi = |f_2/f_1| = |-19.75/44.23| = 0.447$ (*Eq.* B2.3-1)

$k = 4 + 2(1+\psi)^3 + 2(1+\psi)$ (*Eq.* B2.3-2)

$= 4 + 2(1+0.447)^3 + 2(1+0.447) = 12.95$

$f = f_1$

$w = 2.030 - (2)(0.125 + 0.030) = 1.720$ in.

$F_{cr} = 12.95 \dfrac{\pi^2(29500)}{12(1-0.3^2)}\left(\dfrac{0.030}{1.720}\right)^2 = 105.0$ ksi (*Eq.* B2.1-5)

$\lambda = \sqrt{\dfrac{f}{F_{cr}}}$ (*Eq.* B2.1-4)

$\lambda = \sqrt{\dfrac{44.23}{105.0}} = 0.649 < 0.673$; therefore, $b_e = w$

$b_e = w = 1.720$ in. (*Eq.* B2.1-1)

$h_o/b_o = 2.030/2.890 = 0.702 < 4.0$

$b_1 = b_e/(3+\psi)$ (*Eq.* B2.3-3)

$= 1.720/(3+0.447) = 0.499$ in.

For $\psi > 0.236$

$b_2 = b_e/2$ (*Eq.* B2.3-4)

$= 1.720/2 = 0.860$ in.

$w_c = 1.344 - 0.030 - 0.125 = 1.189$ in. (compression portion of web)

$b_1 + b_2 = 0.499 + 0.860 = 1.359$ in. > 1.189 in.

Thus element 5 is fully effective so properties above are correct.

2. Moment of inertia for deflection determination - compression on top, $M_s = 0.6M_n = 7.56$ kip-in.

A conservative approximation of flexural deflections can be obtained by performing an elastic beam analysis using the effective moment of inertia of the cross-section calculated with the maximum extreme fiber stress set to the maximum flexural stress occurring under serviceability loading. In the case of continuous beams, the average of the moments of inertia in maximum positive and negative bending can be used.

For computation of a first approximation of I_{eff}, assume a compressive stress of $f = 0.6F_y = 30$ ksi in the top fibers of the section. Since all elements except 2, 3 and 9 were fully effective at 50 ksi, they will still be fully effective at this lower stress level. Check elements 2, 3 and 9.

Element 2 from Section B4(a)

Use the same simplifying assumption for the complex stiffener lip that was used above.

$w = 3.000 - 3(0.125 + 0.030/2) = 2.580$ in.

$f = 30$ ksi

$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/30} = 40.14$ (*Eq.* B4-7)

$w/t = 2.580/0.030 = 86.0 > 0.328S \therefore$ check effective width

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \quad (Eq.\ B4\text{-}8)$$

$$= 399(0.030)^4\left[\frac{86.0}{40.14} - 0.328\right]^3 \le (0.030)^4\left[115\frac{86.0}{40.14} + 5\right]$$

$I_a = 0.00193$ in.$^4 > 0.000204$ in.$^4 \therefore I_a = 0.000204$ in.^{4}

$I_s = 0.000166$ in.^{4} from above

$R_I = I_s/I_a \le 1$ (*Eq.* B4-9)

$= 0.000166/0.000204 = 0.814$

$$n = \left(0.582 - \frac{w/t}{4S}\right) \ge \frac{1}{3} \quad (Eq.\ B4\text{-}11)$$

$$= \left(0.582 - \frac{86.0}{4(40.14)}\right) \ge \frac{1}{3}$$

$= 0.046 < 1/3 \quad \therefore n = 1/3$

$k = 3.57(0.814)^{1/3} + 0.43 = 3.76$ (From Table B4-1)

$$F_{cr} = 3.76\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.030}{2.580}\right)^2 = 13.56 \text{ ksi} \quad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{30.0}{13.56}} = 1.487 > 0.673 \quad \therefore \text{ element is subject to local buckling} \quad (Eq.\ B2.1\text{-}4)$$

$\lambda_c = 0.256 + 0.328(w/t)\sqrt{F_y/E}$ (*Eq.* B2.1-11)

$= 0.256 + 0.328(2.580/0.030)\sqrt{50/29500} = 1.417$

for $\lambda \ge \lambda_c$,

$\rho = \left(0.41 + 0.59\sqrt{F_y/f_d} - 0.22/\lambda\right)/\lambda$ (*Eq.* B2.1-9)

$= \left(0.41 + 0.59\sqrt{50/30} - 0.22/1.487\right)/1.487 = 0.688$

$b = \rho w$ (*Eq.* B2.1-2)

$= (0.688)(2.580) = 1.775$ in.

Elements 3 and 9 from Section B4(a)

$w = 2.000 - 2(0.125 + 0.030/2) = 1.720$ in.

$f = 30$ ksi

$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/30} = 40.14$ (*Eq.* B4-7)

$w/t = 1.720/0.030 = 57.33 > 0.328S \therefore$ check effective width

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right] \quad (Eq.\ B4\text{-}8)$$

$$= 399(0.030)^4\left[\frac{57.33}{40.14} - 0.328\right]^3 \le (0.030)^4\left[115\frac{57.33}{40.14} + 5\right]$$

$= 0.000430$ in.$^4 > 0.000137$ in.^{4} $\therefore$ $I_a = 0.000137$ in.^{4}

$I_s = 0.0000439$ in.^{4} from above

$$R_I = I_s/I_a = 0.0000439/0.000137 = 0.320 \quad (Eq.\ B4\text{-}9)$$

$$n = \left(0.582 - \frac{57.33}{4(40.14)}\right) \ge \frac{1}{3} \quad (Eq.\ B4\text{-}11)$$

$= 0.225 < 1/3 \quad \therefore n = 1/3$

$D/w = 0.415/1.72 = 0.241 < 0.25 \ \therefore$

$$k = 3.57(0.320)^{1/3} + 0.43 = 2.87 \quad \text{(From Table B4-1)}$$

$$F_{cr} = 2.87\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.030}{1.720}\right)^2 = 23.28 \text{ ksi} \quad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} = \sqrt{\frac{30.0}{23.28}} = 1.135 > 0.673 \ \therefore \text{ element is subject to local buckling} \quad (Eq.\ B2.1\text{-}4)$$

$$\lambda_c = 0.256 + 0.328(1.720/0.030)\sqrt{50/29500} = 1.030 \quad (Eq.\ B2.1\text{-}11)$$

for $\lambda \ge \lambda_c$

$$\rho = \left(0.41 + 0.59\sqrt{50/30} - 0.22/1.135\right)/1.135 = 0.862 \quad (Eq.\ B2.1\text{-}9)$$

$$b = \rho w = (0.862)(1.720) = 1.483 \text{ in.} \quad (Eq.\ B2.1\text{-}2)$$

Effective width of element 9

$$d_s = d'_s(R_I) = (0.260)(0.320) = 0.083 \text{ in.} \quad (Eq.\ B4\text{-}6)$$

Effective section properties about x-axis

Element	L (in.)	y from top fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_x about own axis (in.^{3})
1	1.160	0.321	0.372	0.120	0.031
2	1.775	0.015	0.027	---	---
3	1.483	0.015	0.022	---	---
4	0.660	0.066	0.044	0.003	0.001
5	3.440	1.015	3.492	3.544	0.848
6	4.788	2.015	9.648	19.440	---
7	2.396	1.840	4.409	8.112	0.032
8	2.068	2.015	4.167	8.397	---
9	0.083	0.197	0.016	0.003	---
10	0.440	1.964	0.864	1.697	0.001
Sum $\sum$	18.293		23.061	41.316	0.913

$$\bar{y} = \sum Ly/\sum L = 23.061/18.293 = 1.261 \text{ in.}$$

$$I_e = \left[\sum Ly^2 + \sum I'_x - \bar{y}^2\sum L\right]t$$

$$= \left[41.316 + 0.913 - (1.261)^2(18.293)\right](0.030) = 0.394 \text{ in.}^4$$

$S_e = I_e/\bar{y} = 0.394/1.261 = 0.312 \text{ in.}^3$

$M_n = S_e f = (0.312)(30) = 9.36$ kip-in $> M_s = 7.56$ kip-in.

For the second approximation, estimate the compression stress f in the top fibers of the section at M = 7.56 kip-in. by extrapolation:

M = 12.6 kip-in. at f = 50 ksi

M = 9.36 kip-in. at f = 30 ksi

for M =7.56 kip-in.:

$(12.6 - 9.36)/(50 - 30) = (9.36 - 7.56)/(30 - f)$

f = 18.9 ksi

For the second approximation, assume a compression stress of f = 18.9 ksi in the top fiber of the section.

Element 2 from Section B4(a)

w = 2.580 in.

f = 18.9 ksi

$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/18.9} = 50.57$ (*Eq.* B4-7)

$w/t = 2.580/0.030 = 86.0 > 0.328S \ \therefore$ check local buckling

$I_a = 399t^4\left[\dfrac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\dfrac{w/t}{S} + 5\right]$ (*Eq.* B4-8)

$I_a = 399(0.030)^4\left[\dfrac{86.0}{50.57} - 0.328\right]^3 \le (0.030)^4\left[115\dfrac{86.0}{50.57} + 5\right]$

$= 0.000836 \text{ in.}^4 > 0.000162 \text{ in.}^4$; therefore, $I_a = 0.000162 \text{ in.}^4$

$I_s = 0.000166 \text{ in.}^4$ from above

$R_I = I_s/I_a \le 1$, by inspection $R_I = 1.0$ (*Eq.* B4-9)

$k = 3.57(1.0)^{1/3} + 0.43 = 4.0$ (From Table B4-1)

$F_{cr} = 4.0\dfrac{\pi^2(29500)}{12(1-0.3^2)}\left(\dfrac{0.030}{2.580}\right)^2 = 14.42$ ksi (*Eq.* B2.1-5)

$\lambda = \sqrt{\dfrac{18.9}{14.42}} = 1.145 > 0.673 \ \therefore$ element is subject to local buckling (*Eq.* B2.1-4)

$\lambda_c = 1.417$ (from above)

for $0.673 < \lambda < \lambda_c$

$\rho = (1.358 - 0.461/\lambda)/\lambda$ (*Eq.* B2.1-8)

$= (1.358 - 0.461/1.145)/1.145 = 0.834$

$b = \rho w = (0.834)(2.580) = 2.152$ in. (*Eq.* B2.1-2)

Elements 3 and 9 from Section B4(a)

w = 1.720 in.

f = 18.9 ksi

$S = 1.28\sqrt{E/f} = 1.28\sqrt{29500/18.9} = 50.57$ (*Eq.* B4-7)

$w/t = 1.720/0.030 = 57.33 > 0.328S \ \therefore$ check local buckling

$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \le t^4\left[115\frac{w/t}{S} + 5\right]$ (*Eq.* B4-8)

$= 399(0.030)^4\left[\frac{57.33}{50.57} - 0.328\right]^3 \le (0.030)^4\left[115\frac{57.33}{50.57} + 5\right]$

$= 0.000169 \text{ in.}^4 > 0.000110 \text{ in.}^4 \therefore I_a = 0.000110 \text{ in.}^4$

$R_I = I_s/I_a = 0.0000439/0.000110 = 0.399$ (*Eq.* B4-9)

$n = \left(0.582 - \frac{57.33}{4(50.57)}\right) \ge \frac{1}{3}$ (*Eq.* B4-11)

$= 0.299 < 1/3 \quad \therefore n = 1/3$

$k = 3.57(0.399)^{1/3} + 0.43 = 3.06$ (From Table B4-1)

$F_{cr} = 3.06\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.030}{1.720}\right)^2 = 24.82 \text{ ksi}$ (*Eq.* B2.1-5)

$\lambda = \sqrt{\frac{f}{F_{cr}}} = \sqrt{\frac{18.9}{24.82}} = 0.873 > 0.673 \therefore$ element is subject to local buckling (*Eq.* B2.1-4)

λ_c = 1.030 from above (*Eq.* B2.1-11)

for $0.673 < \lambda < \lambda_c$

$\rho = (1.358 - 0.461/\lambda)/\lambda$ (*Eq.* B2.1-8)

$= (1.358 - 0.461/0.873)/0.873 = 0.951$

$b = \rho w = (0.951)(1.720) = 1.636 \text{ in.}$ (*Eq.* B2.1-2)

Effective width of element 9

$d_s = d'_s(R_I) = (0.260)(0.399) = 0.104 \text{ in.}$ (*Eq.* B4-6)

Effective section properties about x-axis

Element	L (in.)	y from top fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_x about own axis (in.^{3})
1	1.160	0.321	0.372		0.031
2	2.152	0.015	0.032	---	---
3	1.636	0.015	0.025	---	---
4	0.660	0.066	0.044	0.003	0.001
5	3.440	1.015	3.492	3.544	0.848
6	4.788	2.015	9.648	19.440	---
7	2.396	1.840	4.409	8.112	0.032
8	2.068	2.015	4.167	8.397	---
9	0.104	0.207	0.022	0.004	---
10	0.440	1.964	0.864	1.697	0.001
Sum $\sum$	18.844		23.075	41.317	0.913

$\bar{y} = \sum Ly/\sum L = 23.075/18.844 = 1.225 \text{ in.}$

$I_e = \left[\sum Ly^2 + \sum I'_x - \bar{y}^2\sum L\right]t$

$= \left[41.317 + 0.913 - (1.225)^2(18.844)\right](0.030) = 0.419 \text{ in.}^4$

$S_e = I_e/\bar{y} = 0.419/1.225 = 0.342 \text{ in.}^3$

$M_n = S_e f = (0.342)(18.9) = 6.46$ kip-in < M_s = 7.56 kip-in.

For the third approximation, estimate the compression stress f in the top fibers of the section at M = 7.56 kip-in. by interpolation:

M = 9.36 kip-in. at f = 30 ksi

M = 6.46 kip-in. at f = 18.9 ksi

for M =7.56 kip-in.:

$$(9.36-6.46)/(30-18.9) = (9.36-7.56)/(30-f)$$

f = 23.1 ksi

Repeating the previous calculations (not shown) with f =23.1 ksi gives:

I_e = 0.419 in.^{4}

S_e = 0.342 in.^{3}

M = S_ef = (0.342)(23.1) = 7.90 kip-in. ≅ 7.56 kip-in.

Therefore, the effective moment of inertia is:

I_e = 0.419 in.^{4}

3. Section modulus, S_e, for nominal flexural strength - compression on bottom

If the neutral axis is closer to the compression flange than to the tension flange, the compression stress is less than F_y and is unknown, and therefore, the effective width of the compression flange and the effective section properties must be determined by an iterative method. By inspection, elements 1, 2, 3, 4, and 9 are in tension and are therefore fully effective. Assume compression stress will govern, i.e., f = F_y = 50 ksi in the bottom compression fibers of the section.

Elements 6, 7 and 8 from Section B5.1

Check the effective width of the intermediately stiffened elements at the bottom of the panel per Section B5.1.1 for the case of two identical stiffeners, equally spaced.

n = 2

$A_g = (4.788+2.396+2.068)(0.030) = 0.278$ in.^{2} (from Example I-7)

$A_s = (1.198)(0.030) = 0.0359$ in.^{2}

$b_o = 3.000+3.000+3.000-(2)(0.125+0.030/2) = 8.720$ in.

$h = 2.030-2(0.125+0.030) = 1.720$ in.

$$I_{sp} = I'_x t + A_s y^2$$

$$= (0.0159)(0.030)+(0.0359)(0.380/2-0.030/2)^2 = 0.00158 \text{ in.}^4$$

$$\gamma = \frac{10.92 I_{sp}}{b_o t^3} \quad (Eq.\ B5.1.1\text{-}4)$$

$$= \frac{(10.92)(0.00158)}{(8.720)(0.030)^3} = 73.3$$

$$\delta = \frac{A_s}{b_o t} \quad (Eq.\ B5.1.1\text{-}5)$$

$$= \frac{0.0359}{(8.72)(0.030)} = 0.137$$

$$\beta = (1+\gamma(n+1))^{1/4} \quad (Eq.\ B5.1.1\text{-}3)$$

$$= (1+73.3(2+1))^{1/4} = 3.86$$

$$k_d = \frac{\left(1+\beta^2\right)^2 + \gamma\left(1+n\right)}{\beta^2\left(1+\delta\left(n+1\right)\right)} \qquad (Eq.\ B5.1.1\text{-}2)$$

$$= \frac{\left(1+3.86^2\right)^2 + 73.3\left(1+2\right)}{3.86^2\left(1+0.137\left(2+1\right)\right)} = 22.5$$

$b_o/h = 8.720/1.72 = 5.07 > 1.0$; therefore,

$$R = \frac{11 - b_o/h}{5} \geq \frac{1}{2} \qquad (Eq.\ B5.1\text{-}6)$$

$$= \frac{11 - 8.720/1.720}{5} = 1.186$$

$$Rk_d = \left(1.186\right)\left(22.5\right) = 26.7$$

$$k_{loc} = 4\left(n+1\right)^2 \qquad (Eq.\ B5.1.1\text{-}1)$$

$$= 4\left(2+1\right)^2 = 36.0$$

$k = \min\left(Rk_d,\ k_{loc}\right) = \min\left(26.7, 36.0\right) = 26.7 \ \therefore$ distortional buckling controls

$$F_{cr} = k\frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{b_o}\right)^2 \qquad (Eq.\ B5.1\text{-}5)$$

$$= 26.7\frac{\pi^2\left(29500\right)}{12\left(1-0.3^2\right)}\left(\frac{0.030}{8.720}\right)^2 = 8.43 \text{ ksi}$$

$\lambda = \sqrt{\frac{f}{F_{cr}}} = \sqrt{\frac{50}{8.43}} = 2.435 > 0.673 \ \therefore$ element is subject to buckling (*Eq*. B5.1-4)

$$\rho = \left(1-0.22/\lambda\right)/\lambda \qquad (Eq.\ B5.1\text{-}3)$$

$$= \left(1-0.22/2.435\right)/2.435 = 0.374$$

$$b_e = \rho\left(\frac{A_g}{t}\right) \qquad (Eq.\ B5.1\text{-}1)$$

$$= 0.374\left(\frac{0.278}{0.030}\right) = 3.466 \text{ in.}$$

location of centroid of stiffened element

$$\bar{y} = \frac{\left(4.788\right)\left(2.015\right)+\left(2.396\right)\left(1.840\right)+\left(2.068\right)\left(2.015\right)}{4.788+2.396+2.068} = 1.970 \text{ in.}$$

Effective section properties about x-axis, assuming element 5 is fully effective:

Element	L (in.)	y from top fiber (in.)	Ly (in.^{2})	Ly^2 (in.^{3})	I'_x about own axis (in.^{3})
1	1.160	0.321	0.372	0.120	0.031
2	2.580	0.015	0.039	0.001	---
3	1.720	0.015	0.026	---	---
4	0.660	0.066	0.044	0.003	0.001
5	3.440	1.015	3.492	3.544	0.848
6, 7, 8	3.466	1.970	6.828	13.451	---
9	0.260	0.285	0.074	0.021	0.001
10	0.440	1.964	0.864	1.697	0.001
Sum $\sum$	13.726		11.739	18.837	0.882

$\bar{y} = \sum Ly/\sum L = 11.739/13.726 = 0.855$ in. < 1.015 in. (above centerline)

$y_c = 2.030 - 0.855 = 1.175$ in.

Therefore, compression stress controls as assumed.

$I_e = \left[\sum Ly^2 + \sum I'_x - \bar{y}^2 \sum L\right] t$

$= \left[18.837 + 0.882 - (0.855)^2 (13.726)\right](0.030) = 0.291 \text{ in.}^4$

$S_{et} = I_e/\bar{y} = 0.291/0.855 = 0.340 \text{ in.}^3$

$S_{eb} = 0.291/1.175 = 0.248 \text{ in.}^3$

$M_n = \min(S_{et}, S_{eb})F_y = 0.248(50) = 12.4$ kip-in. (*Eq.* C3.1.1-1)

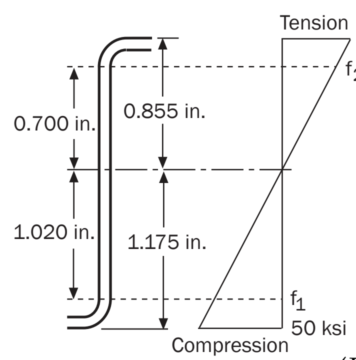
Element 5 from Section B2.3(a): check assumption that element is fully effective

$f_1 = \left[(1.175 - 0.125 - 0.030)/1.175\right](50)$

$= 43.40$ ksi

$f_2 = -\left[(0.855 - 0.125 - 0.030)/1.175\right](50)$

$= -29.79$ ksi



$\psi = |f_2 / f_1| = |-29.79/43.40| = 0.686$ (*Eq.* B2.3-1)

$k = 4 + 2(1+\psi)^3 + 2(1+\psi)$ (*Eq.* B2.3-2)

$= 4 + 2(1+0.686)^3 + 2(1+0.686) = 16.96$

$f = f_1$

$w = 1.720$ in.

$F_{cr} = 16.96 \dfrac{\pi^2 (29500)}{12(1-0.3^2)}\left(\dfrac{0.030}{1.720}\right)^2 = 137.6$ ksi (*Eq.* B2.1-5)

$\lambda = \sqrt{\dfrac{f}{F_{cr}}}$ (*Eq.* B2.1-4)

$= \sqrt{\dfrac{43.40}{137.6}} = 0.562 < 0.673$

$b_e = w = 1.720$ in. (*Eq.* B2.1-1)

$h_o/b_o = 2.03/8.72 = 0.23 < 4$

$b_1 = b_e/(3+\psi)$ (*Eq.* B2.3-3)

$= 1.720/(3+0.686) = 0.467$ in.

For $\psi > 0.236$

$b_2 = b_e/2$ (*Eq.* B2.3-4)

$= 1.720/2 = 0.860$ in.

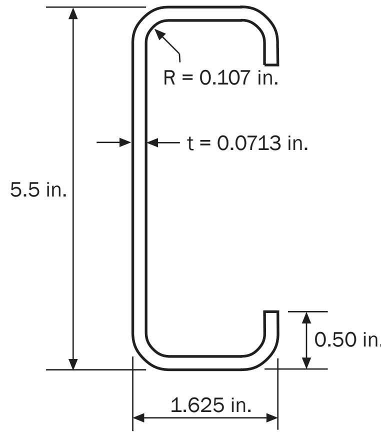
$w_c = 1.175 - 0.030 - 0.125 = 1.020$ in. (compression portion of web)

$b_1 + b_2 = 0.467 + 0.860 = 1.327$ in. > 1.020 in.

Thus element 5 is fully effective so properties above are correct.

3.8 Special Topics

Example I-15: Strength Increase From Cold Work Of Forming



Given:

1. $F_{yv} = F_y = 33$ ksi
2. $F_{uv} = F_u = 45$ ksi
3. Section: SSMA Stud 550S162-68 as shown above
4. Section to be used as a beam and is fully braced against lateral-torsional buckling

Required:

1. Determine the nominal flexural strength, M_n, considering the increase in strength resulting from the cold work of forming, using the provisions of *Specification* Section A7.2.

Solution:

1. Check the limitations

In order to use *Eq.* A7.2-1 for computing the average tensile yield stress for the beam flange, the geometry of the section and yield stress must be such that the quantity ρ is unity as determined according to Section B2 for each of the flat elements of the section. In the case of webs under a stress gradient, if the sum of b_1 and b_2 from Section B2.3 at least equals the depth of the compression portion of the web, the web is considered to meet this criteria, even if ρ is less than 1.0.

Assume ρ = 1.0 for each flat element and check the elements at a maximum flexural stress, f, of F_{ya}.

Eq. A7.2-2 is applicable only when $F_{uv}/F_{yv} \geq 1.2$, $R/t \leq 7$ and the minimum included angle ≤ 120°.

$F_{uv}/F_{yv} = 45/33 = 1.36 > 1.2$ OK

$R/t = 0.107/0.0713 = 1.50 \leq 7$ OK

$\theta = 90° < 120°$ OK

Therefore, *Eq.* A7.2-2 can be used to determine F_{yc}

2. Calculation of F_{yc}

$$B_c = 3.69\left(F_{uv}/F_{yv}\right) - 0.819\left(F_{uv}/F_{yv}\right)^2 - 1.79 \qquad (Eq.\ A7.2\text{-}3)$$

$$= 3.69(1.36) - 0.819(1.36)^2 - 1.79 = 1.714$$

$$m = 0.192\left(F_{uv}/F_{yv}\right) - 0.068 \qquad (Eq.\ A7.2\text{-}4)$$

$$= 0.192(1.36) - 0.068 = 0.193$$

$F_{yc} = B_c F_{yv} / (R/t)^m$ (*Eq.* A7.2-2)

$= 1.714(33)/(1.50)^{0.193} = 52.31 \text{ ksi}$

3. Calculation of F_{ya}

$r = R + t/2 = 0.107 + 0.0713/2 = 0.143 \text{ in.}$

Cross-sectional area of corner = $(\pi/2)(0.143)(0.0713) = 0.0160 \text{ in.}^2$

Total corner cross-sectional area of the controlling flange

$= (0.0160)(2) = 0.0320 \text{ in.}^2$

Flat width of the compression flange

$w = b - 2(t + R)$

$= 1.625 - 2(0.0713 + 0.107) = 1.268 \text{ in.}$

Full cross-sectional area of the controlling flange

$A_{flange} = 0.0320 + (1.268)(0.0713) = 0.122 \text{ in.}^2$

$C = 0.0320/0.122 = 0.262$

$F_{ya} = CF_{yc} + (1 - C)F_{yf}$ (*Eq.* A7.2-1)

$= (0.262)(52.31) + (1 - 0.262)(33) = 38.06 \text{ ksi}$

4. Check effective width assumptions

Recheck each flat compression element based on an extreme fiber compressive bending stress of 38.06 ksi. It can be demonstrated, by calculations not shown, that each flat element is fully effective, therefore, the increase from cold work of forming may be used.

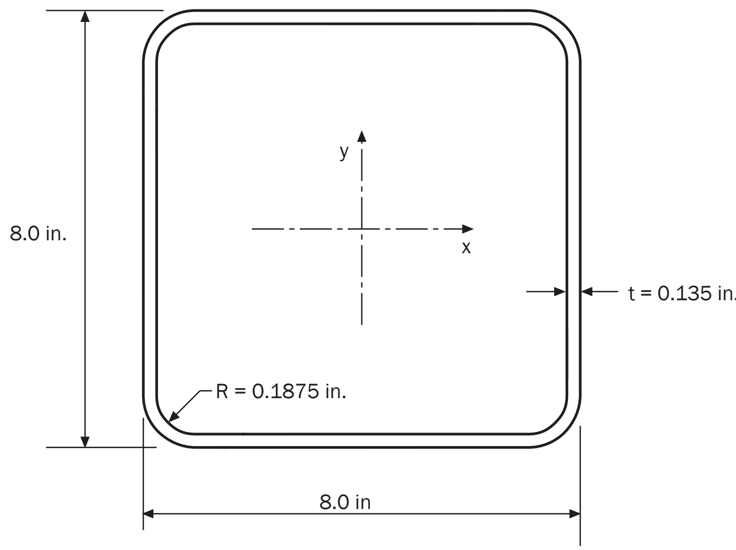
5. Calculation of M_n

$M_n = S_e F_y$ (*Eq.* C3.1.1-1)

where F_y is taken as F_{ya} per Section A7.2

$M_n = (1.041)(38.06) = 39.6 \text{ kip-in.}$

Example I-16: Shear Lag



Given:

1. Steel: F_y = 50 ksi
2. Section: 8 x 8 x 0.135 square tube
3. Span: L = 3 ft., with simple supports
4. Loading: Concentrated load at midspan
5. A_{gross} = 4.19 in.^{2}
6. I_{gross} = 42.8 in.^{4}

Required:

1. Determine the ASD flexural allowable strength, M_n/Ω_b
2. Determine the LRFD flexural design strength, $\phi_b M_n$

Solution:

Compute the nominal flexural strength, M_n, as the lesser of the values determined according to Sections C3.1 and B1.1(c).

1. Nominal moment strength, M_n, based on initiation of yielding (Section C3.1.1)

Since the member is not subject to lateral-torsional buckling, compute the nominal strength using Section C3.1.1.

Check compression flange in accordance with Section B2.1 with f = 50 ksi and k = 4.00. The compression flange is found to have an effective width of 5.071 in., by calculations not shown.

Check webs in accordance with Section B2.3. The reduced effective width of the compression flange will cause the neutral axis to shift towards the tension flange. Using f_1 and f_2 for the new position of the neutral axis, the webs are found to be fully effective, by calculation not shown.

The net section properties can then be calculated as:

I_x = 37.7 in.^{4}
S_e = 8.74 in.^{3}
M_n= 437 kip-in.

2. Nominal moment strength, M_n, considering shear lag (Section B1.1(c))

$w_f = \left[8.0-(2)(0.135)\right]/2 = 3.865\text{ in.}$

L/w_f= (3)(12)/3.865 = 9.31 < 30

Because the L/w_f ratio is less than 30, and the member carries a concentrated load, consideration of shear lag is required.

Interpolating from Table B1.1(c):

for $L/w_f = 10$, effective design width/actual width = 0.73
for $L/w_f = 8$, effective design width/actual width = 0.67
for $L/w_f = 9.31$,

$$\text{effective design width/actual width} = 0.67 + \left(\frac{0.73-0.67}{10-8}\right)(9.31-8) = 0.71$$

Therefore, the maximum effective design widths of the compression and tension flanges between the inside of the webs are $0.71\left[8.0-(2)(0.135)\right]=5.488$ in.

The effective flat width of the flanges is:

$$b_{max} = 5.488 - 2R = 5.488 - (2)(0.1875) = 5.113 \text{ in.}$$

Recalculate properties using effective compression and tension flange widths of 5.113 in. (governed by Section B1.1).

$I_x = 33.43 \text{ in.}^4$
$S_e = 8.36 \text{ in.}^3$
$M_n = (8.36)(50) = 418$ kip-in. CONTROLS

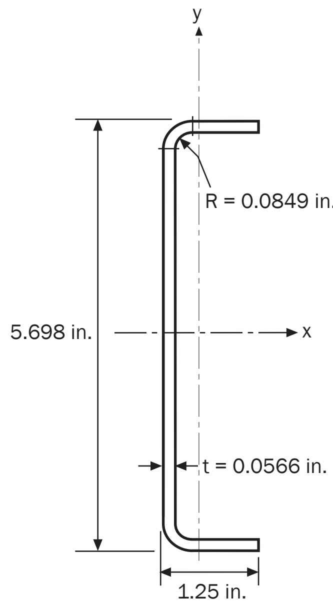
3. Determination of the ASD flexural allowable design strength:

$M \le M_n/\Omega_b$ (*Eq.* A4.1.1-1)
$\Omega_b = 1.67$
$M \le 418/1.67 = 250$ kip-in.

4. Determination of the LRFD flexural design strength:

$M_u \le \phi_b M_n$ (*Eq.* A5.1.1-1)
$\phi_b = 0.95$
$M_u \le (0.95)(418) = 397$ kip-in.

Example I-17: Flange Curling



Given:

1. Steel: F_y = 33 ksi
2. Section: SSMA Track 550T125-54 as shown above
3. Compression flange braced against lateral buckling

Required:

1. Determine the amount of curling of the compression flange at a maximum flexural compressive stress of 30.93 ksi, as used in Example II-3.

Solution:

1. Average stress in compression flange, f_{av}

From Example I-9

w = 1.109 in.

b = 0.854 in.

$f_{av} = f(b/w) = 30.93(0.854/1.109) = 23.82$ ksi

2. Curling of the compression flange, c_f

$w_f = 1.250 - 0.0566 = 1.193$ in.

$$w_f = \sqrt{0.061tdE/f_{av}}\sqrt[4]{(100c_f/d)} \qquad (Eq.\ B1.1\text{-}1)$$

Solving for c_f:

$$c_f = \frac{w_f^{\,4}}{100d\left[0.061tE/f_{av}\right]^2}$$

$$= \frac{1.193^4}{(100)(5.698)\left[(0.061)(0.0566)(29500)/23.82\right]^2}$$

c_f = 0.000194 in.

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PART II
BEAM DESIGN

For Use With the
2007 Edition of the
North American Specification for the Design
of Cold-Formed Steel Structural Members

PART II - BEAM DESIGN

Strength design of cold-formed steel beams requires the consideration of the limit states of:

1. Flexural yielding and local buckling
2. Lateral-torsional buckling
3. Distortional buckling
4. Web shear yielding or buckling, including interaction with flexure
5. Web crippling, including interaction with flexure
6. Combined flexure and torsion

Specification Section C3 includes provisions for the evaluation of these limit states. For beams that are parts of certain structural systems, Section D6 provides specific provisions for those members. In addition to these limit states, consideration of bracing and anchorage forces is necessary in some cases. For shear lag effects, see *Specification* Section B1.1(c).

Yielding and Local Buckling: The strength of all flexural members is limited by the combined limit state of yielding and local buckling, which is evaluated using Section C3.1.1(a). Yielding strength is calculated using the effective section modulus, S_e. For members in which all elements are fully effective, additional strength can be calculated considering the effects of the cold work of forming using Section A7.2. For flexural members not subject to twisting or lateral, torsional or flexural-torsional buckling, the strength may alternately be calculated based on the inelastic reserve capacity using Section C3.1.1(b). See *Manual* Section 3.6 and Examples I-8 through I-10 and I-12 through I-15 in Part I for further information on the calculation of effective section properties and the yielding strength of flexural members.

Lateral-Torsional Buckling: General provisions for calculating the lateral-torsional buckling strength of unbraced or discretely braced members are found in *Specification* Section C3.1.2. Although the specifics vary somewhat for different cross-section shapes, the general procedure involves 1) determination of the elastic lateral-torsional buckling stress, 2) transformation of the elastic buckling stress to a critical buckling stress, taking into account the effects of inelasticity and 3) determination of the effective section modulus with the extreme compression fiber at the critical buckling stress. Section D6 of the *Specification* provides specialized provisions for the lateral-torsional buckling of flexural members that are elements of metal roof and wall systems, including through-fastened purlins and girts, and standing seam roofs.

Distortional Buckling: The distortional buckling limit state involves the cross-sectional deformations of two or more elements acting as a group, e.g., the rotation of the flange and lip of a C-shape about the web-to-flange junction. The *Specification* provides three levels of provisions for this limit state in Section C3.1.4. Section C3.1.4(a) requires a simple calculation using basic cross-section dimensions and produces a conservative, and sometimes very conservative, result. This approach can sometimes be used to quickly determine that distortional buckling is not a controlling limit state. For those cases where the extra work is justified, Section C3.1.4(b) can be used, which requires considerably more complex calculations, but produces accurate results. Section C3.1.4(c) provides a framework for the use of computerized numerical methods to evaluate distortional buckling. This approach requires fewer calculations than Section C3.1.4(b) and is especially useful for cross-sections that do not meet the limits of applicability of the other two approaches. For all three approaches, the general procedure involves 1) determination of the elastic distortional buckling stress, 2) determination of the corresponding elastic buckling moment using the gross section modulus of the cross-section and 3) transformation of the elastic buckling moment to a nominal flexural strength, taking into account the effects of inelasticity and post-buckling strength.

Shear Strength: The shear limit state checks are similar to those for hot-rolled shapes. Depending upon the slenderness of the web, the shear strength may be limited by yielding, inelastic buckling or elastic buckling. Section C3.2 contains provisions for the shear strength of webs with and without holes. The combined limit state of flexural yielding and shear is evaluated according to the provisions of Section C3.3.

Web Crippling: Web crippling of cold-formed members is a localized buckling and yielding of the web in the vicinity of a concentrated load. The web crippling strength is a function of the load bearing length, web thickness and height, bend radius, yield strength and member geometry. Strengths are determined by Section C3.4 with the use of empirically derived coefficients provided in a series of tables. For C-sections with web holes in the vicinity of the concentrated load, a strength reduction factor must be applied. The combined limit state of flexure and web crippling is evaluated using Section C3.5.

Torsion: Flexural members not loaded through the shear center generate torsional stresses in the cross section unless they are braced against torsion. Section C3.6 contains provisions for reducing the flexural strength due to torsional warping. Although C-shapes loaded in their strong axis through the flange would be expected to exhibit torsion, it is not considered necessary to apply these provisions to C-shapes used in metal roof and wall systems when determining system strength using the provisions of Section D6.

SECTION 1 - BENDING

1.1 Notes On The Tables

(a) With the exception of the SSMA studs and tracks, the specific sections listed in these tables are not necessarily stock sections. They are included primarily as a guide in the design of cold-formed steel structural members.

(b) The section designations listed in these tables correspond to those for which dimensions and properties are given in Tables I-1 to I-5 and I-8.

(c) The effective section modulus values are calculated as the effective moment of inertia at the indicated stress level divided by the distance to the controlling extreme fiber. In calculating the nominal strength of these sections, additional checks such as the provisions of Chapter C of the *Specification* should also be taken into account where applicable.

(d) Tabulated section properties are shown to three significant figures, while dimensions are given to three decimal places.

(e) Where they apply, the algebraic formulae presented in Section 3 of Part I formed the basis of the calculations for these tables.

(f) The effective section properties listed in Tables II-1 through II-6 were computed using the yield stresses listed in the tables, except where a value of F_{ya} is given in Tables II-2 and II-3, the provisions of Section A7 of the *Specification* for strength increase from cold work of forming have been used. Sections were considered eligible for the cold work of forming increase in yield stress if ρ=1.0 for each flat element, except that webs may have ρ<1.0 if the sum of b_1 plus b_2 from Section B2.3 equals or exceeds the width of the compression portion of the web.

(g) The values labeled I_e in Tables II-1 through II-6 are effective moments of inertia calculated at nominal moments that are 60 percent of M_{nxo}. They represent lower bound values of I_x for use in estimating deflections at ordinary service loads.

(h) The values in the columns labeled M_{web}, M_{flange} and M_{lip} in Tables II-1 through II-5 are the highest nominal moments at which the web, flange and lip (if applicable) respectively are fully effective. These values may be used to determine if each of the elements is fully effective at a given nominal moment. These values are only meaningful where they do not exceed M_{nxo} for the section and yield stress in question.

(i) The effects of standard factory punchouts in SSMA studs have been included in Table II-2. These punchouts are considered in SSMA studs with flange widths of 1.625 in. or less. Standard punchout sizes are 1.5 in. by 4.5 in. for sections with depths of 3.5 inches or more and 0.75 in. by 4.5 in. for sections with shallower depths. Punchouts are not included in the calculations for sections with web h/t ratios in excess of 200 due to the limits in Section B2.4. Those sections are marked with a "*".

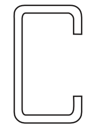
(j) Dashes in the place of data values in the F_{ya} columns of Table II-2 indicate that the section is not eligible for strength increase due to the effects of cold work of forming for the listed yield stress. Dashes in other columns indicate that the section is not available in the listed grade of steel.

1.2 Beam Property Tables

Table II - 1

Beam Properties[3]
C-Sections With Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.95 (LRFD)
Ω_v = 1.60 (ASD)
ϕ_v = 0.95 (LRFD)

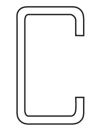


Section	F_y = 33 ksi				F_y = 55 ksi				Maximum Fully-Effective Moment [2]		
	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	M_{web} kip-in.	M_{flange} kip-in.	M_{lip} kip-in.
12CS4x105	14.4	246	7.45	47.5	14.4	368	6.70	46.3	414	234	159
12CS4x085	7.63	187	5.65	38.1	7.63	264	4.79	36.5	230	135	97.9
12CS4x070	4.25	143	4.32	31.0	4.25	189	3.44	29.6	134	81.8	64.3
12CS3.5x105	14.4	234	7.10	43.8	14.4	358	6.51	43.4	423	286	201
12CS3.5x085	7.63	183	5.53	35.6	7.63	261	4.74	34.9	235	166	123
12CS3.5x070	4.25	142	4.29	29.2	4.25	188	3.42	28.2	138	101	80.8
12CS2.5x105	14.4	196	5.93	36.3	14.4	309	5.62	36.0	156	425	353
12CS2.5x085	7.63	152	4.61	29.4	7.63	232	4.22	28.8	82.3	244	210
12CS2.5x070	4.25	120	3.64	23.8	4.25	169	3.07	23.4	45.8	148	134
10CS4x105	15.1	192	5.82	31.0	17.5	286	5.19	30.3	404	183	125
10CS4x085	9.25	145	4.40	25.0	9.25	220	4.01	23.5	224	106	76.8
10CS4x070	5.15	115	3.48	20.2	5.15	158	2.87	19.1	131	64.2	50.5
10CS4x065	4.12	105	3.18	18.4	4.12	140	2.54	17.7	107	53.1	43.3
10CS3.5x105	15.1	183	5.54	28.5	17.5	277	5.04	28.2	417	223	157
10CS3.5x085	9.25	142	4.31	23.1	9.25	214	3.89	22.6	232	129	96.3
10CS3.5x070	5.15	111	3.38	19.0	5.15	158	2.86	18.1	136	79.1	63.1
10CS3.5x065	4.12	102	3.10	17.6	4.12	139	2.53	16.7	112	65.7	54.0
10CS2.5x105	15.1	154	4.66	23.3	17.5	256	4.66	23.3	450	373	298
10CS2.5x085	9.25	125	3.79	19.0	9.25	204	3.70	19.0	251	217	180
10CS2.5x070	5.15	104	3.14	15.7	5.15	153	2.78	15.7	147	133	117
10CS2.5x065	4.12	96.2	2.92	14.6	4.12	134	2.43	14.6	121	110	99.8
10CS2x105	15.1	137	4.15	20.7	17.5	221	4.02	20.7	159	461	434
10CS2x085	9.25	109	3.29	16.9	9.25	172	3.13	16.7	83.7	267	256
10CS2x070	5.15	86.3	2.61	13.9	5.15	136	2.47	13.6	46.5	163	161
10CS2x065	4.12	78.9	2.39	12.8	4.12	124	2.26	12.5	37.2	134	136
9CS2.5x105	15.1	133	4.02	18.1	19.5	221	4.02	18.1	439	321	257
9CS2.5x085	9.88	108	3.27	14.7	10.3	176	3.19	14.7	243	187	156
9CS2.5x070	5.76	89.4	2.71	12.2	5.76	136	2.47	12.2	143	114	101
9CS2.5x065	4.60	83.1	2.52	11.3	4.60	121	2.21	11.3	116	95.0	86.2
9CS2.5x059	3.44	74.7	2.26	10.3	3.44	104	1.89	10.3	90.2	74.6	70.3
8CS4x105	15.1	143	4.34	18.6	19.5	211	3.83	18.1	389	137	93.1
8CS4x085	9.88	108	3.26	15.0	11.7	162	2.95	14.1	216	79.2	57.5
8CS4x070	6.52	84.8	2.57	12.1	6.52	127	2.31	11.3	126	48.1	37.9
8CS4x065	5.22	77.4	2.34	11.1	5.22	112	2.03	10.4	103	39.8	32.4
8CS4x059	3.90	68.7	2.08	9.93	3.90	95.1	1.73	9.41	79.6	31.2	26.6
8CS3.5x105	15.1	136	4.11	16.9	19.5	205	3.72	16.8	405	165	117
8CS3.5x085	9.88	105	3.19	13.8	11.7	157	2.86	13.5	225	96.2	71.7
8CS3.5x070	6.52	82.3	2.49	11.4	6.52	124	2.26	10.7	133	58.9	47.1
8CS3.5x065	5.22	75.3	2.28	10.5	5.22	112	2.04	9.84	109	48.9	40.3
8CS3.5x059	3.90	67.2	2.04	9.41	3.90	95.4	1.74	8.90	83.9	38.4	33.0
8CS2.5x105	15.1	113	3.41	13.6	19.5	188	3.41	13.6	431	272	218
8CS2.5x085	9.88	91.8	2.78	11.1	11.7	149	2.71	11.1	240	159	132
8CS2.5x070	6.52	76.0	2.30	9.21	6.52	115	2.10	9.21	142	97.2	85.9
8CS2.5x065	5.22	70.7	2.14	8.57	5.22	105	1.91	8.57	117	80.7	73.3
8CS2.5x059	3.90	63.5	1.92	7.79	3.90	93.0	1.69	7.79	90.3	63.4	59.8

Table II - 1

Beam Properties[3]
C-Sections With Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.95 (LRFD)
Ω_v = 1.60 (ASD)
ϕ_v = 0.95 (LRFD)



Section	F_y = 33 ksi				F_y = 55 ksi				Maximum Fully-Effective Moment [2]		
	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	M_{web} kip-in.	M_{flange} kip-in.	M_{lip} kip-in.
8CS2x105	15.1	99.1	3.00	12.0	19.5	165	3.00	12.0	465	379	351
8CS2x085	9.88	80.8	2.45	9.79	11.7	135	2.45	9.79	261	222	210
8CS2x070	6.52	66.9	2.03	8.11	6.52	111	2.03	8.11	155	137	135
8CS2x065	5.22	62.2	1.89	7.54	5.22	104	1.89	7.54	127	113	115
8CS2x059	3.90	56.6	1.72	6.86	3.90	93.3	1.70	6.86	98.6	85.3	92.7
7CS4x105	13.3	121	3.66	13.7	19.5	176	3.21	13.4	380	115	78.5
7CS4x085	9.88	90.4	2.74	11.1	12.8	135	2.46	10.4	210	66.8	48.5
7CS4x070	6.70	71.0	2.15	8.97	7.53	106	1.93	8.32	123	40.6	32.0
7CS4x065	5.78	64.8	1.96	8.17	6.02	96.7	1.76	7.64	101	33.6	27.4
7CS4x059	4.49	57.4	1.74	7.34	4.49	83.1	1.51	6.88	77.4	26.3	22.5
7CS2.5x105	13.3	93.7	2.84	9.94	19.5	156	2.84	9.94	429	226	181
7CS2.5x085	9.88	76.5	2.32	8.11	12.8	124	2.26	8.11	239	132	110
7CS2.5x070	6.70	63.4	1.92	6.72	7.53	95.8	1.74	6.72	141	81.0	71.6
7CS2.5x065	5.78	59.0	1.79	6.25	6.02	87.3	1.59	6.25	116	67.3	61.2
7CS2.5x059	4.49	53.0	1.61	5.69	4.49	78.4	1.42	5.69	89.9	52.9	49.9
6CS4x105	11.3	99.2	3.01	9.64	18.8	144	2.62	9.44	368	94.7	64.5
6CS4x085	9.18	74.1	2.25	7.81	12.8	110	2.00	7.34	204	55.0	40.0
6CS4x070	6.70	58.1	1.76	6.36	8.65	86.1	1.57	5.87	119	33.5	26.4
6CS4x065	5.78	52.9	1.60	5.78	7.12	78.5	1.43	5.40	97.3	27.8	22.6
6CS4x059	4.76	46.9	1.42	5.19	5.31	69.5	1.26	4.83	74.9	21.7	18.5
6CS2.5x105	11.3	76.0	2.30	6.91	18.8	127	2.30	6.91	426	184	147
6CS2.5x085	9.18	62.1	1.88	5.65	12.8	101	1.83	5.65	237	107	89.5
6CS2.5x070	6.70	51.5	1.56	4.69	8.65	77.7	1.41	4.69	140	65.9	58.3
6CS2.5x065	5.78	48.0	1.45	4.36	7.12	70.7	1.29	4.36	115	54.8	49.8
6CS2.5x059	4.76	43.1	1.31	3.97	5.31	63.4	1.15	3.97	89.0	43.1	40.6
4CS4x105	7.10	59.9	1.81	3.87	11.8	85.7	1.56	3.81	341	57.3	38.8
4CS4x085	5.82	44.5	1.35	3.16	9.69	65.2	1.19	2.98	187	33.4	24.2
4CS4x070	4.83	34.7	1.05	2.60	8.05	50.6	0.921	2.38	109	20.4	16.0
4CS4x065	4.50	31.6	0.956	2.35	7.46	46.0	0.837	2.19	88.7	16.9	13.7
4CS4x059	4.10	27.9	0.844	2.11	6.15	40.6	0.739	1.95	68.2	13.3	11.3
4CS2.5x105	7.10	44.1	1.34	2.67	11.8	73.5	1.34	2.67	416	107	85.3
4CS2.5x085	5.82	36.3	1.10	2.20	9.69	59.1	1.08	2.20	229	62.9	52.3
4CS2.5x070	4.83	30.3	0.917	1.83	8.05	45.4	0.825	1.83	134	38.8	34.2
4CS2.5x065	4.50	28.2	0.855	1.71	7.46	41.2	0.750	1.71	110	32.3	29.3
4CS2.5x059	4.10	25.4	0.771	1.56	6.15	36.9	0.671	1.56	84.8	25.4	23.9
4CS2x105	7.10	37.9	1.15	2.30	11.8	63.2	1.15	2.30	433	137	155
4CS2x085	5.82	31.3	0.947	1.89	9.69	52.1	0.947	1.89	244	80.2	94.8
4CS2x070	4.83	26.1	0.791	1.58	8.05	43.5	0.791	1.58	146	47.5	61.8
4CS2x065	4.50	24.3	0.737	1.47	7.46	40.4	0.734	1.47	121	38.8	52.6
4CS2x059	4.10	22.2	0.673	1.35	6.15	36.1	0.656	1.35	94.5	29.6	43.0

Notes:

1. Shear and moment strengths given are nominal strengths. To obtain available strengths, these values must be modified by safety factors (ASD) or resistance factors (LRFD).
2. M_{web}, M_{flange} and M_{lip} are the highest nominal moments at which the web, flange and lip, respectively, are fully effective.

3. The distortional buckling limit state is not considered in this table. Consideration of distortional buckling may result in lower strengths when restraint against distortional buckling is not provided. See Table II-7 for distortional buckling strengths.

Table II - 2

Beam Properties[4]
SSMA Studs
C-Sections With Lips

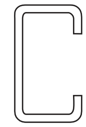
Ω_b = 1.67 (ASD)
ϕ_b = 0.95 (LRFD)
Ω_v = 1.60 (ASD)
ϕ_v = 0.95 (LRFD)

Section	F_y = 33 ksi, F_u = 45 ksi					F_y = 50 ksi, F_u = 65 ksi					Maximum Fully-Effective Moment [2]		
	V_n [1] kips	F_{ya} [3] ksi.	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	V_n [1] kips	F_{ya} [3] ksi	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	M_{web} kip-in.	M_{flange} kip-in.	M_{lip} kip-in.
1200S250-97	13.0	-	181	5.50	34.0	13.0	-	252	5.04	33.8	134	310	203
1200S250-68	4.43	-	115	3.50	24.0	4.43	-	150	3.01	23.6	45.7	127	97.7
1200S250-54*	2.20	-	81.9	2.48	18.8	2.20	-	107	2.15	18.4	22.7	72.2	63.5
1200S200-97	13.0	-	162	4.90	30.4	13.0	-	233	4.66	30.2	120	419	299
1200S200-68	4.43	-	106	3.22	21.4	4.43	-	148	2.96	20.9	41.0	167	138
1200S200-54*	2.20	-	80.0	2.43	16.7	2.20	-	104	2.07	16.3	20.4	94.4	87.6
1200S162-97	11.9	-	143	4.33	27.0	11.9	-	205	4.09	26.7	107	491	335
1200S162-68	4.43	-	93.0	2.82	19.0	4.43	-	132	2.64	18.4	36.5	189	142
1200S162-54*	2.20	-	69.6	2.11	14.7	2.20	-	95.7	1.91	14.3	18.2	105	86.5
1000S250-97	14.1	37.7	164	4.36	21.8	15.8	56.2	235	4.18	21.8	371	268	163
1000S250-68	5.35	36.4	110	3.03	15.8	5.35	-	138	2.77	15.7	135	113	82.5
1000S250-54	2.66	-	75.1	2.28	12.7	2.66	-	94.0	1.88	12.7	72.5	65.2	55.7
1000S250-43*	1.34	-	53.4	1.62	10.2	-	-	-	-	-	41.5	37.8	38.7
1000S200-97	14.1	-	128	3.87	19.3	15.8	-	187	3.74	19.3	136	342	242
1000S200-68	5.35	-	86.0	2.61	13.9	5.35	-	121	2.42	13.7	46.0	137	112
1000S200-54	2.66	-	65.5	1.98	11.0	2.66	-	85.3	1.70	10.8	22.8	77.5	71.7
1000S200-43*	1.34	-	48.5	1.47	8.60	-	-	-	-	-	11.5	42.3	46.8
1000S162-97	10.3	-	112	3.39	17.0	11.5	-	163	3.27	17.0	119	401	271
1000S162-68	5.35	-	75.1	2.28	12.3	5.35	-	108	2.15	12.0	40.5	155	116
1000S162-54	2.66	-	56.8	1.72	9.63	2.66	-	78.6	1.57	9.39	20.2	86.1	70.8
1000S162-43*	1.34	-	43.0	1.30	7.52	-	-	-	-	-	10.2	49.2	45.4
800S250-97	14.1	37.7	120	3.19	12.8	17.4	56.2	172	3.05	12.8	362	195	119
800S250-68	6.75	36.4	80.7	2.22	9.26	6.75	-	103	2.06	9.24	131	82.9	60.6
800S250-54	3.34	-	56.5	1.71	7.47	3.34	-	76.3	1.52	7.38	71.0	47.9	41.0
800S250-43	1.68	-	43.3	1.31	6.02	-	-	-	-	-	40.9	27.8	28.5
800S200-97	14.1	38.8	109	2.80	11.2	17.4	57.6	161	2.80	11.2	382	281	186
800S200-68	6.75	37.2	75.6	2.04	8.14	6.75	55.4	109	1.96	8.14	138	116	90.6
800S200-54	3.34	36.3	59.7	1.64	6.57	3.34	-	74.9	1.50	6.57	74.1	66.3	60.1
800S200-43	1.68	-	42.7	1.29	5.30	-	-	-	-	-	42.4	36.3	40.9
800S200-33*	0.758	-	26.9	0.816	4.10	-	-	-	-	-	19.3	16.4	23.7
800S162-97	7.72	40.1	97.3	2.43	9.71	9.50	-	121	2.43	9.71	139	311	208
800S162-68	5.39	-	57.3	1.74	7.09	5.39	-	83.2	1.66	7.07	46.8	121	89.3
800S162-54	3.34	-	44.0	1.33	5.70	3.34	-	61.4	1.23	5.60	23.2	67.4	55.2
800S162-43	1.68	-	33.6	1.02	4.50	-	-	-	-	-	11.7	38.6	35.6
800S162-33*	0.758	-	23.4	0.710	3.38	-	-	-	-	-	5.29	19.5	21.5
800S137-97	7.72	41.2	88.7	2.15	8.60	9.50	-	107	2.15	8.60	123	330	213
800S137-68	5.39	-	50.9	1.54	6.30	5.39	-	73.4	1.47	6.28	41.6	127	82.7
800S137-54	3.34	-	38.9	1.18	5.08	3.34	-	54.2	1.08	4.97	20.7	69.7	48.6
800S137-43	1.68	-	29.6	0.896	4.00	-	-	-	-	-	10.4	39.2	30.1
800S137-33*	0.758	-	20.5	0.622	3.00	-	-	-	-	-	4.72	20.6	17.6
600S250-97	11.1	37.7	81.5	2.16	6.50	16.8	56.2	116	2.06	6.50	354	132	80.9
600S250-68	6.96	36.4	54.8	1.51	4.73	8.56	-	69.3	1.39	4.72	126	56.3	41.3
600S250-54	4.38	-	38.2	1.16	3.82	4.52	-	53.4	1.07	3.76	67.9	32.7	28.0
600S250-43	2.26	-	30.3	0.918	3.08	-	-	-	-	-	39.0	19.0	19.5

Table II - 2

Beam Properties[4]
SSMA Studs
C-Sections With Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.95 (LRFD)
Ω_v = 1.60 (ASD)
ϕ_v = 0.95 (LRFD)

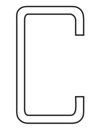


Section	F_y = 33 ksi, F_u = 45 ksi					F_y = 50 ksi, F_u = 65 ksi					Maximum Fully-Effective Moment [2]		
	V_n [1] kips	F_{ya} [3] ksi.	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	V_n [1] kips	F_{ya} [3] ksi	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	M_{web} kip-in.	M_{flange} kip-in.	M_{lip} kip-in.
600S200-97	11.1	38.8	72.6	1.87	5.61	16.8	57.6	108	1.87	5.61	373	187	125
600S200-68	6.96	37.2	50.8	1.37	4.10	8.56	55.4	73.0	1.32	4.10	132	77.6	60.9
600S200-54	4.38	36.3	40.2	1.11	3.32	4.52	-	50.8	1.01	3.32	71.1	44.6	40.5
600S200-43	2.26	-	28.8	0.873	2.68	-	-	-	-	-	41.3	24.5	27.6
600S200-33	1.02	-	20.5	0.621	2.06	-	-	-	-	-	19.9	11.1	18.2
600S162-97	4.02	40.1	64.1	1.60	4.80	6.09	59.2	94.7	1.60	4.80	384	228	143
600S162-68	3.74	38.1	44.7	1.17	3.52	4.61	56.6	65.9	1.16	3.52	131	91.7	63.7
600S162-54	3.02	37.1	35.3	0.954	2.86	3.12	55.3	50.6	0.916	2.86	67.4	51.9	40.6
600S162-43	1.98	36.3	27.9	0.767	2.32	-	-	-	-	-	36.7	30.1	27.1
600S162-33	1.02	-	19.1	0.577	1.79	-	-	-	-	-	19.5	15.2	17.2
600S137-97	4.02	41.2	57.6	1.40	4.19	6.09	60.8	84.8	1.40	4.19	150	234	148
600S137-68	3.74	38.9	40.2	1.03	3.09	4.61	-	51.5	1.03	3.09	50.8	91.5	58.3
600S137-54	3.02	-	27.5	0.832	2.52	3.12	-	38.8	0.777	2.52	25.0	50.5	34.6
600S137-43	1.98	-	21.3	0.645	2.04	-	-	-	-	-	12.6	28.5	21.6
600S137-33	1.02	-	15.0	0.455	1.55	-	-	-	-	-	5.70	15.0	12.8
550S162-68	3.29	38.1	39.6	1.04	2.86	4.05	56.6	58.4	1.03	2.86	129	81.2	56.4
550S162-54	2.66	37.1	31.3	0.845	2.32	3.01	55.3	44.9	0.811	2.32	66.5	45.9	36.0
550S162-43	1.92	36.3	24.7	0.681	1.88	-	-	-	-	-	36.5	26.7	24.1
550S162-33	1.12	-	16.9	0.512	1.46	-	-	-	-	-	19.4	13.5	15.2
400S200-68	5.14	37.2	29.5	0.795	1.59	7.79	55.4	42.4	0.766	1.59	125	45.1	35.4
400S200-54	4.17	36.3	23.5	0.646	1.29	5.39	-	29.4	0.589	1.29	65.8	26.1	23.6
400S200-43	2.78	-	16.8	0.509	1.05	-	-	-	-	-	37.4	14.3	16.1
400S200-33	1.56	-	12.0	0.365	0.804	-	-	-	-	-	19.4	6.49	11.4
400S162-68	1.43	-	21.7	0.658	1.33	2.17	-	32.4	0.648	1.32	0.00	48.7	35.0
400S162-54	1.51	-	17.3	0.526	1.08	1.96	-	24.9	0.498	1.07	0.00	27.5	22.1
400S162-43	1.30	-	13.7	0.417	0.870	-	-	-	-	-	0.00	16.0	14.6
400S162-33	0.952	-	9.86	0.299	0.669	-	-	-	-	-	0.00	8.12	9.03
400S137-68	1.43	-	18.7	0.568	1.14	2.17	-	27.9	0.558	1.14	0.00	50.5	31.7
400S137-54	1.51	-	15.0	0.453	0.934	1.96	-	21.4	0.428	0.929	0.00	28.1	19.1
400S137-43	1.30	-	11.8	0.359	0.754	-	-	-	-	-	0.00	16.1	12.1
400S137-33	0.952	-	8.54	0.259	0.580	-	-	-	-	-	0.00	8.57	7.25
362S200-68	4.61	37.2	25.9	0.698	1.27	6.99	55.4	37.3	0.673	1.26	123	39.6	31.1
362S200-54	3.74	36.3	20.6	0.568	1.03	5.39	-	25.9	0.517	1.03	64.4	22.9	20.8
362S200-43	2.78	-	14.8	0.448	0.836	-	-	-	-	-	36.3	12.6	14.2
362S200-33	1.64	-	10.6	0.321	0.642	-	-	-	-	-	19.2	5.72	10.3
362S162-68	1.06	-	19.1	0.579	1.05	1.61	-	28.7	0.574	1.05	0.00	43.4	31.0
362S162-54	1.13	-	15.4	0.466	0.857	1.63	-	22.2	0.444	0.854	0.00	24.6	19.7
362S162-43	1.08	-	12.3	0.372	0.694	-	-	-	-	-	0.00	14.3	13.0
362S162-33	0.834	-	8.84	0.268	0.535	-	-	-	-	-	0.00	7.26	8.08
362S137-68	1.06	-	16.4	0.498	0.902	1.61	-	24.7	0.493	0.902	0.00	45.0	28.1
362S137-54	1.13	-	13.3	0.402	0.740	1.63	-	19.1	0.381	0.737	0.00	25.1	16.9
362S137-43	1.08	-	10.6	0.320	0.600	-	-	-	-	-	0.00	14.4	10.8
362S137-33	0.834	-	7.66	0.232	0.462	-	-	-	-	-	0.00	7.68	6.49

Table II - 2

Beam Properties[4]
SSMA Studs
C-Sections With Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.95 (LRFD)
Ω_v = 1.60 (ASD)
ϕ_v = 0.95 (LRFD)



Section	F_y = 33 ksi, F_u = 45 ksi					F_y = 50 ksi, F_u = 65 ksi					Maximum Fully-Effective Moment [2]		
	V_n [1] kips	F_{ya} [3] ksi.	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	V_n [1] kips	F_{ya} [3] ksi	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	M_{web} kip-in.	M_{flange} kip-in.	M_{lip} kip-in.
350S162-68	0.947	-	18.2	0.551	0.964	1.44	-	27.5	0.549	0.965	0.00	41.6	29.7
350S162-54	1.01	-	14.7	0.447	0.788	1.52	-	21.3	0.426	0.787	0.00	23.6	18.9
350S162-43	1.01	-	11.8	0.357	0.640	-	-	-	-	-	0.00	13.7	12.5
350S162-33	0.779	-	8.49	0.257	0.494	-	-	-	-	-	0.00	6.98	7.76
250S162-68	0.548	38.1	13.7	0.360	0.450	0.830	56.6	20.2	0.357	0.450	121	28.1	19.5
250S162-54	0.596	37.1	11.0	0.296	0.370	0.903	55.3	15.7	0.284	0.370	59.7	16.1	12.6
250S162-43	0.631	36.3	8.72	0.240	0.302	-	-	-	-	-	31.6	9.43	8.50
250S162-33	0.638	-	5.94	0.180	0.235	-	-	-	-	-	16.5	4.80	5.38
250S137-68	0.548	38.9	12.0	0.309	0.386	0.830	57.8	17.8	0.308	0.386	126	29.3	17.7
250S137-54	0.596	37.8	9.62	0.254	0.318	0.903	56.2	13.7	0.244	0.318	60.1	16.7	10.8
250S137-43	0.631	36.9	7.56	0.205	0.261	-	-	-	-	-	30.7	9.68	7.01
250S137-33	0.638	-	5.20	0.158	0.203	-	-	-	-	-	15.3	5.21	4.31

Notes:

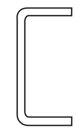
* Web h/t > 200, therefore bearing stiffeners are required.

1. Shear and moment strengths given are nominal strengths. To obtain available strengths, these values must be modified by safety factors (ASD) or resistance factors (LRFD).
2. M_{web}, M_{flange} and M_{lip} are the highest nominal moments at which the web, flange and lip, respectively, are fully effective.
3. Where values are given for F_{ya}, flexural strength is calculated using the strength increase from cold working, based on an average yield stress of F_{ya}.
4. The distortional buckling limit state is not considered in this table. Consideration of distortional buckling may result in lower strengths when restraint against distortional buckling is not provided. See Table II-8 for distortional buckling strengths.

Table II - 3

Beam Properties
SSMA Tracks
C-Sections Without Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.95 (LRFD)
Ω_v = 1.60 (ASD)
ϕ_v = 0.95 (LRFD)

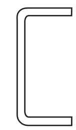


Section	F_y = 33 ksi, F_u = 45 ksi				F_y = 50 ksi, F_u = 65 ksi				Maximum Fully-Effective Moment [2]	
	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	M_{web} kip-in.	M_{flange} kip-in.
1200T200-97	12.6	142	4.30	29.8	12.6	191	3.82	29.0	104	85.1
1200T200-68	4.34	78.7	2.38	19.3	4.34	103	2.06	18.0	34.8	27.0
1200T200-54*	2.17	52.2	1.58	14.1	2.17	67.5	1.35	13.0	17.2	13.0
1200T150-97	12.6	132	4.00	26.0	12.6	181	3.62	25.7	93.8	137
1200T150-68	4.34	75.3	2.28	17.6	4.34	99.3	1.99	16.6	31.8	42.8
1200T150-54*	2.17	50.5	1.53	13.0	2.17	65.7	1.31	12.0	15.8	20.5
1200T125-97	12.6	122	3.69	24.1	12.6	172	3.44	23.7	86.9	184
1200T125-68	4.34	72.8	2.21	16.2	4.34	96.7	1.93	15.7	29.5	56.2
1200T125-54*	2.17	49.2	1.49	12.3	2.17	64.3	1.29	11.5	14.6	26.6
1000T200-97	14.1	113	3.43	19.0	15.2	154	3.08	18.6	108	65.0
1000T200-68	5.22	63.9	1.94	12.5	5.22	84.2	1.68	11.8	35.9	20.6
1000T200-54	2.60	42.8	1.30	9.23	2.60	55.6	1.11	8.56	17.7	9.94
1000T200-43*	1.32	28.4	0.861	6.72	-	-	-	-	8.84	4.90
1000T150-97	14.1	104	3.17	16.4	15.2	145	2.90	16.4	103	109
1000T150-68	5.22	60.9	1.85	11.3	5.22	81.0	1.62	10.8	34.7	33.8
1000T150-54	2.60	41.2	1.25	8.43	2.60	53.9	1.08	7.88	17.1	16.0
1000T150-43*	1.32	27.6	0.837	6.19	-	-	-	-	8.56	7.83
1000T125-97	14.1	95.9	2.91	15.1	15.2	138	2.75	15.1	94.7	147
1000T125-68	5.22	58.8	1.78	10.5	5.22	78.7	1.58	10.2	32.0	44.9
1000T125-54	2.60	40.1	1.22	7.96	2.60	52.8	1.06	7.48	15.9	21.2
1000T125-43*	1.32	27.0	0.819	5.88	-	-	-	-	7.99	10.3
800T200-97	14.1	82.2	2.49	11.2	17.4	117	2.35	10.8	112	47.3
800T200-68	6.54	49.2	1.49	7.30	6.54	65.5	1.31	7.05	37.0	15.0
800T200-54	3.26	33.3	1.01	5.50	3.26	43.6	0.872	5.15	18.1	7.23
800T200-43	1.65	22.3	0.676	4.04	-	-	-	-	9.04	3.57
800T200-33*	0.744	14.0	0.424	2.79	-	-	-	-	4.08	1.60
800T150-97	14.1	74.9	2.27	9.48	17.4	110	2.19	9.48	111	78.6
800T150-68	6.54	46.6	1.41	6.53	6.54	62.8	1.25	6.36	36.5	24.2
800T150-54	3.26	32.0	0.969	4.97	3.26	42.2	0.844	4.69	17.9	11.5
800T150-43	1.65	21.6	0.655	3.69	-	-	-	-	8.92	5.61
800T150-33*	0.744	13.7	0.414	2.57	-	-	-	-	4.03	2.52
800T125-97	14.1	77.8[3]	2.06	8.61	17.4	103	2.06	8.61	107	111
800T125-68	6.54	44.8	1.36	6.00	6.54	60.8	1.22	5.96	35.6	33.4
800T125-54	3.26	31.0	0.940	4.67	3.26	41.2	0.824	4.42	17.5	15.7
800T125-43	1.65	21.1	0.640	3.48	-	-	-	-	8.73	7.60
800T125-33*	0.744	13.4	0.407	2.44	-	-	-	-	3.94	3.41
600T200-97	11.8	55.0	1.67	5.76	17.4	78.4	1.57	5.56	251	32.0
600T200-68	6.96	34.1	1.03	3.70	8.56	48.6	0.973	3.54	80.5	10.2
600T200-54	4.36	25.1	0.759	2.76	4.36	35.9	0.717	2.64	38.9	4.89
600T200-43	2.20	18.6	0.565	2.08	-	-	-	-	19.2	2.41
600T200-33	0.996	11.0	0.333	1.54	-	-	-	-	8.68	1.08

Table II - 3

Beam Properties
SSMA Tracks
C-Sections Without Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.95 (LRFD)
Ω_v = 1.60 (ASD)
ϕ_v = 0.95 (LRFD)

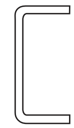


Section	F_y = 33 ksi, F_u = 45 ksi				F_y = 50 ksi, F_u = 65 ksi				Maximum Fully-Effective Moment [2]	
	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	M_{web} kip-in.	M_{flange} kip-in.
600T150-97	11.8	49.6	1.50	4.78	17.4	72.2	1.44	4.78	118	52.1
600T150-68	6.96	31.8	0.963	3.26	8.56	44.6	0.891	3.16	38.4	16.0
600T150-54	4.36	22.8	0.689	2.47	4.36	30.5	0.609	2.40	18.6	7.60
600T150-43	2.20	15.6	0.474	1.89	-	-	-	-	9.25	3.71
600T150-33	0.996	10.0	0.303	1.33	-	-	-	-	4.18	1.67
600T150-30	0.730	8.36	0.253	1.16	-	-	-	-	3.07	1.22
600T150-27*	0.545	7.06	0.214	1.01	-	-	-	-	2.29	0.910
600T125-97	11.8	50.8[3]	1.35	4.28	17.4	67.4	1.35	4.28	117	73.0
600T125-68	6.96	30.2	0.916	2.97	8.56	42.9	0.858	2.93	38.1	21.9
600T125-54	4.36	22.0	0.666	2.30	4.36	29.6	0.592	2.24	18.5	10.2
600T125-43	2.20	15.2	0.461	1.77	-	-	-	-	9.19	4.97
600T125-33	0.996	9.80	0.297	1.26	-	-	-	-	4.15	2.23
600T125-30	0.730	8.22	0.249	1.09	-	-	-	-	3.05	1.63
600T125-27*	0.545	6.95	0.210	0.958	-	-	-	-	2.27	1.21
550T200-68	6.96	30.2	0.914	3.03	8.56	42.9	0.857	2.89	79.1	9.07
550T200-54	4.38	22.1	0.669	2.25	4.77	31.5	0.630	2.15	38.1	4.36
550T200-43	2.41	16.3	0.495	1.69	-	-	-	-	18.8	2.15
550T200-33	1.09	10.1	0.307	1.25	-	-	-	-	8.49	0.968
550T150-68	6.96	28.1	0.850	2.66	8.56	40.2	0.804	2.57	87.6	14.2
550T150-54	4.38	20.7	0.628	2.00	4.77	29.7	0.595	1.93	42.1	6.74
550T150-43	2.41	15.4	0.468	1.52	-	-	-	-	20.8	3.29
550T150-33	1.09	10.2	0.310	1.11	-	-	-	-	9.37	1.48
550T150-30	0.798	8.29	0.251	0.994	-	-	-	-	6.87	1.08
550T150-27	0.595	6.85	0.208	0.892	-	-	-	-	5.12	0.807
550T125-68	6.96	26.6	0.807	2.41	8.56	38.4	0.769	2.38	38.7	19.3
550T125-54	4.38	19.7	0.597	1.86	4.77	26.7	0.535	1.81	18.8	9.05
550T125-43	2.41	13.7	0.416	1.43	-	-	-	-	9.31	4.39
550T125-33	1.09	8.90	0.270	1.03	-	-	-	-	4.21	1.97
550T125-30	0.798	7.47	0.226	0.897	-	-	-	-	3.08	1.44
550T125-27	0.595	6.32	0.192	0.786	-	-	-	-	2.30	1.07
400T200-68	5.50	19.5	0.591	1.49	8.33	27.4	0.549	1.41	73.4	6.06
400T200-54	4.38	14.1	0.426	1.09	5.39	19.8	0.397	1.04	35.1	2.91
400T200-43	2.78	10.3	0.311	0.810	-	-	-	-	17.2	1.43
400T200-33	1.50	7.25	0.220	0.581	-	-	-	-	7.76	0.646
400T150-68	5.50	18.1	0.548	1.29	8.33	25.6	0.513	1.24	83.5	9.29
400T150-54	4.38	13.2	0.399	0.960	5.39	18.7	0.374	0.918	39.7	4.41
400T150-43	2.78	9.68	0.293	0.719	-	-	-	-	19.4	2.15
400T150-33	1.50	6.88	0.208	0.519	-	-	-	-	8.76	0.967
400T150-30	1.10	6.03	0.183	0.458	-	-	-	-	6.42	0.708
400T150-27	0.823	5.08	0.154	0.409	-	-	-	-	4.79	0.528

Table II - 3

Beam Properties
SSMA Tracks
C-Sections Without Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.95 (LRFD)
Ω_v = 1.60 (ASD)
ϕ_v = 0.95 (LRFD)

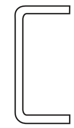


Section	F_y = 33 ksi, F_u = 45 ksi				F_y = 50 ksi, F_u = 65 ksi				Maximum Fully-Effective Moment [2]	
	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	M_{web} kip-in.	M_{flange} kip-in.
400T125-68	5.50	17.1	0.517	1.15	8.33	24.4	0.488	1.13	89.7	12.4
400T125-54	4.38	12.6	0.381	0.882	5.39	17.9	0.359	0.849	42.6	5.83
400T125-43	2.78	9.30	0.282	0.666	-	-	-	-	20.8	2.83
400T125-33	1.50	6.63	0.201	0.484	-	-	-	-	9.38	1.27
400T125-30	1.10	5.82	0.176	0.427	-	-	-	-	6.87	0.929
400T125-27	0.823	5.14	0.156	0.380	-	-	-	-	5.13	0.692
400T125-18*	0.242	2.32	0.0701	0.241	-	-	-	-	1.50	0.202
362T200-68	4.97	17.1	0.519	1.20	7.52	24.0	0.480	1.14	71.6	5.37
362T200-54	3.97	12.3	0.372	0.879	5.39	17.3	0.345	0.832	34.1	2.58
362T200-43	2.78	8.92	0.270	0.649	-	-	-	-	16.7	1.27
362T200-33	1.64	6.28	0.190	0.464	-	-	-	-	7.53	0.572
362T150-68	4.97	15.9	0.480	1.03	7.52	22.4	0.449	0.993	82.1	8.19
362T150-54	3.97	11.5	0.349	0.769	5.39	16.3	0.325	0.734	38.9	3.88
362T150-43	2.78	8.42	0.255	0.574	-	-	-	-	19.0	1.90
362T150-33	1.64	5.95	0.180	0.414	-	-	-	-	8.56	0.853
362T150-30	1.22	5.21	0.158	0.364	-	-	-	-	6.27	0.624
362T150-27	0.910	4.60	0.140	0.323	-	-	-	-	4.68	0.465
362T125-68	4.97	15.0	0.453	0.921	7.52	21.3	0.427	0.907	88.7	10.9
362T125-54	3.97	11.0	0.332	0.705	5.39	15.6	0.312	0.678	41.9	5.12
362T125-43	2.78	8.08	0.245	0.531	-	-	-	-	20.4	2.48
362T125-33	1.64	5.74	0.174	0.384	-	-	-	-	9.20	1.11
362T125-30	1.22	5.03	0.152	0.339	-	-	-	-	6.75	0.816
362T125-27	0.910	4.45	0.135	0.301	-	-	-	-	5.03	0.608
362T125-18	0.267	2.10	0.0637	0.189	-	-	-	-	1.47	0.177
350T200-68	4.79	16.4	0.496	1.11	7.26	22.9	0.458	1.05	71.0	5.15
350T200-54	3.83	11.7	0.355	0.814	5.39	16.4	0.329	0.770	33.8	2.47
350T200-43	2.78	8.49	0.257	0.600	-	-	-	-	16.5	1.22
350T200-33	1.64	5.97	0.181	0.428	-	-	-	-	7.45	0.549
350T150-68	4.79	15.1	0.459	0.957	7.26	21.4	0.428	0.919	81.5	7.84
350T150-54	3.83	11.0	0.332	0.711	5.39	15.5	0.310	0.679	38.6	3.71
350T150-43	2.78	8.01	0.243	0.530	-	-	-	-	18.8	1.81
350T150-33	1.64	5.66	0.172	0.382	-	-	-	-	8.48	0.816
350T150-30	1.26	4.95	0.150	0.336	-	-	-	-	6.22	0.597
350T150-27	0.943	4.37	0.132	0.298	-	-	-	-	4.64	0.445
350T125-68	4.79	14.3	0.433	0.851	7.26	20.4	0.407	0.839	88.3	10.4
350T125-54	3.83	10.5	0.317	0.651	5.39	14.9	0.297	0.626	41.6	4.89
350T125-43	2.78	7.69	0.233	0.490	-	-	-	-	20.3	2.37
350T125-33	1.64	5.46	0.165	0.354	-	-	-	-	9.14	1.06
350T125-30	1.26	4.78	0.145	0.312	-	-	-	-	6.70	0.779
350T125-27	0.943	4.22	0.128	0.277	-	-	-	-	5.00	0.581
350T125-18	0.277	2.03	0.0615	0.174	-	-	-	-	1.46	0.169
250T200-68	3.38	10.7	0.324	0.548	5.12	14.8	0.296	0.517	64.9	3.48
250T200-54	2.71	7.53	0.228	0.396	4.10	10.4	0.209	0.371	30.5	1.67
250T200-43	2.17	5.37	0.163	0.288	-	-	-	-	14.8	0.820
250T200-33	1.64	3.71	0.112	0.203	-	-	-	-	6.67	0.369

Table II - 3

Beam Properties
SSMA Tracks
C-Sections Without Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.95 (LRFD)
Ω_v = 1.60 (ASD)
ϕ_v = 0.95 (LRFD)



Section	F_y = 33 ksi, F_u = 45 ksi				F_y = 50 ksi, F_u = 65 ksi				Maximum Fully-Effective Moment [2]	
	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	M_{web} kip-in.	M_{flange} kip-in.
250T150-68	3.38	9.88	0.299	0.465	5.12	13.8	0.276	0.445	76.3	5.20
250T150-54	2.71	7.05	0.214	0.342	4.10	9.84	0.197	0.324	35.6	2.46
250T150-43	2.17	5.07	0.154	0.252	-	-	-	-	17.2	1.20
250T150-33	1.64	3.52	0.107	0.180	-	-	-	-	7.75	0.540
250T150-30	1.33	3.06	0.0929	0.157	-	-	-	-	5.68	0.395
250T150-27	1.10	2.69	0.0815	0.139	-	-	-	-	4.24	0.295
250T125-68	3.38	9.28	0.281	0.408	5.12	13.1	0.262	0.402	84.1	6.83
250T125-54	2.71	6.70	0.203	0.310	4.10	9.42	0.188	0.297	39.1	3.20
250T125-43	2.17	4.86	0.147	0.231	-	-	-	-	18.8	1.55
250T125-33	1.64	3.40	0.103	0.166	-	-	-	-	8.48	0.696
250T125-30	1.33	2.96	0.0896	0.145	-	-	-	-	6.21	0.509
250T125-27	1.10	2.60	0.0788	0.129	-	-	-	-	4.63	0.380
250T125-18	0.392	1.46	0.0443	0.0778	-	-	-	-	1.36	0.111
162T125-33	1.06	1.92	0.0583	0.0656	-	-	-	-	7.59	0.418
162T125-30	0.956	1.66	0.0504	0.0574	-	-	-	-	5.56	0.306
162T125-27	0.866	1.45	0.0440	0.0505	-	-	-	-	4.15	0.228
162T125-18	0.484	0.831	0.0252	0.0297	-	-	-	-	1.21	0.0667

Notes:

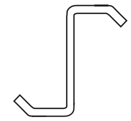
* Web h/t > 200, therefore bearing stiffeners are required.

1. Shear and moment strengths given are nominal strengths. To obtain available strengths, these values must be modified by safety factors (ASD) or resistance factors (LRFD).
2. M_{web} and M_{flange} are the highest nominal moments at which the web and flange, respectively, are fully effective.
3. Flexural properties were calculated incorporating strength increase from cold work of forming with F_{ya} = 37.72 ksi

Table II - 4

Beam Properties[3]
Z-Sections With Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.95 (LRFD)
Ω_v = 1.60 (ASD)
ϕ_v = 0.95 (LRFD)

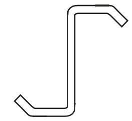


Section	F_y = 33 ksi				F_y = 55 ksi				Maximum Fully-Effective Moment [2]		
	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	M_{web} kip-in.	M_{flange} kip-in.	M_{lip} kip-in.
12ZS3.25x105	14.4	241	7.29	43.7	14.4	364	6.61	43.7	424	325	261
12ZS3.25x085	7.63	190	5.75	35.5	7.63	266	4.84	35.5	242	193	168
12ZS3.25x070	4.25	148	4.49	29.3	4.25	195	3.54	29.2	148	120	114
12ZS2.75x105	14.4	216	6.54	40.0	14.4	339	6.16	39.8	172	377	331
12ZS2.75x085	7.63	168	5.10	32.4	7.63	241	4.38	31.8	90.6	223	208
12ZS2.75x070	4.25	133	4.05	26.3	4.25	180	3.28	25.9	50.4	139	138
12ZS2.25x105	14.4	195	5.92	36.3	14.4	309	5.62	36.0	156	482	461
12ZS2.25x085	7.63	152	4.60	29.4	7.63	239	4.35	28.7	82.2	286	260
12ZS2.25x070	4.25	120	3.64	23.8	4.25	184	3.34	23.2	45.7	163	151
10ZS3.25x105	15.1	188	5.69	28.4	17.5	282	5.13	28.4	416	253	204
10ZS3.25x085	9.25	148	4.48	23.1	9.25	217	3.95	23.1	239	150	131
10ZS3.25x070	5.15	115	3.50	19.1	5.15	163	2.97	18.7	146	93.9	89.3
10ZS3.25x065	4.12	107	3.23	17.8	4.12	145	2.64	17.2	122	78.7	77.7
10ZS3.25x059	3.08	96.3	2.92	16.1	3.08	125	2.28	15.7	96.5	59.8	64.6
10ZS2.75x105	15.1	171	5.17	25.9	17.5	282	5.12	25.9	424	321	274
10ZS2.75x085	9.25	139	4.21	21.0	9.25	213	3.87	21.0	243	191	175
10ZS2.75x070	5.15	115	3.47	17.4	5.15	161	2.93	17.4	150	120	119
10ZS2.75x065	4.12	105	3.17	16.2	4.12	144	2.61	16.2	125	96.5	103
10ZS2.75x059	3.08	93.3	2.83	14.7	3.08	124	2.26	14.6	99.7	72.3	85.2
10ZS2.25x105	15.1	154	4.66	23.3	17.5	249	4.53	23.3	178	392	374
10ZS2.25x085	9.25	122	3.70	18.9	9.25	194	3.53	18.8	93.9	233	216
10ZS2.25x070	5.15	97.4	2.95	15.6	5.15	150	2.73	15.3	52.1	133	126
10ZS2.25x065	4.12	89.2	2.70	14.4	4.12	131	2.38	14.1	41.7	107	102
10ZS2.25x059	3.08	79.4	2.41	13.0	3.08	114	2.08	12.8	31.1	80.4	77.4
9ZS2.25x105	15.1	132	4.01	18.1	19.5	221	4.01	18.1	444	368	345
9ZS2.25x085	9.88	108	3.27	14.7	10.3	180	3.27	14.7	256	220	207
9ZS2.25x070	5.76	89.1	2.70	12.2	5.76	145	2.64	12.2	156	126	121
9ZS2.25x065	4.60	82.9	2.51	11.3	4.60	130	2.36	11.3	129	101	97.9
9ZS2.25x059	3.44	75.3	2.28	10.3	3.44	111	2.01	10.3	101	76.3	74.7
8ZS3.25x105	15.1	139	4.23	16.9	19.5	208	3.79	16.9	400	187	152
8ZS3.25x085	9.88	110	3.33	13.8	11.7	160	2.91	13.8	229	112	97.6
8ZS3.25x070	6.52	85.4	2.59	11.4	6.52	128	2.33	10.9	140	69.9	66.6
8ZS3.25x065	5.22	79.0	2.39	10.6	5.22	117	2.13	10.1	116	58.6	57.9
8ZS3.25x059	3.90	71.2	2.16	9.62	3.90	101	1.83	9.25	92.2	44.5	48.1
8ZS2.75x105	15.1	126	3.82	15.3	19.5	208	3.78	15.3	411	236	202
8ZS2.75x085	9.88	103	3.11	12.4	11.7	156	2.84	12.4	236	141	130
8ZS2.75x070	6.52	84.8	2.57	10.3	6.52	124	2.25	10.3	145	88.5	88.1
8ZS2.75x065	5.22	77.3	2.34	9.56	5.22	114	2.07	9.56	121	71.4	76.0
8ZS2.75x059	3.90	68.9	2.09	8.69	3.90	100	1.82	8.62	96.1	53.6	62.9
8ZS2.25x105	15.1	112	3.41	13.6	19.5	187	3.41	13.6	421	312	293
8ZS2.25x085	9.88	91.6	2.77	11.1	11.7	153	2.77	11.1	240	187	179
8ZS2.25x070	6.52	75.7	2.30	9.18	6.52	124	2.25	9.18	149	107	104
8ZS2.25x065	5.22	70.4	2.13	8.54	5.22	110	2.01	8.54	125	86.2	84.8
8ZS2.25x059	3.90	64.0	1.94	7.76	3.90	98.8	1.80	7.76	99.7	64.9	64.6

Table II - 4

Beam Properties[3]
Z-Sections With Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.95 (LRFD)
Ω_v = 1.60 (ASD)
ϕ_v = 0.95 (LRFD)



Section	F_y = 33 ksi				F_y = 55 ksi				Maximum Fully-Effective Moment [2]		
	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	M_{web} kip-in.	M_{flange} kip-in.	M_{lip} kip-in.
7ZS2.25x105	13.3	93.5	2.83	9.92	19.5	156	2.83	9.92	410	260	244
7ZS2.25x085	9.88	76.3	2.31	8.09	12.8	127	2.31	8.09	236	156	153
7ZS2.25x070	6.70	63.1	1.91	6.70	7.53	103	1.87	6.70	146	89.4	89.2
7ZS2.25x065	5.78	58.7	1.78	6.23	6.02	91.8	1.67	6.23	122	71.9	72.4
7ZS2.25x059	4.49	53.4	1.62	5.67	4.49	82.2	1.49	5.67	97.4	54.2	55.1
6ZS2.25x105	11.3	75.8	2.30	6.89	18.8	126	2.30	6.89	401	211	198
6ZS2.25x085	9.18	61.9	1.88	5.63	12.8	103	1.88	5.63	231	127	126
6ZS2.25x070	6.70	51.3	1.55	4.66	8.65	83.7	1.52	4.66	142	72.6	74.7
6ZS2.25x065	5.78	47.8	1.45	4.34	7.12	74.5	1.36	4.34	119	58.5	60.6
6ZS2.25x059	4.76	43.5	1.32	3.95	5.31	66.6	1.21	3.95	94.4	44.1	46.1
4ZS2.25x070	4.83	30.0	0.910	1.82	8.05	49.0	0.892	1.82	131	42.5	48.7
4ZS2.25x065	4.50	28.0	0.848	1.70	7.46	43.6	0.794	1.70	109	34.3	39.5
4ZS2.25x059	4.10	25.5	0.773	1.55	6.15	39.0	0.708	1.55	86.1	25.9	30.0
3.5ZS1.5x070	4.14	18.6	0.563	0.985	6.90	31.0	0.563	0.985	131	56.8	55.7
3.5ZS1.5x065	3.86	17.3	0.525	0.918	6.42	28.9	0.525	0.918	107	47.6	47.4
3.5ZS1.5x059	3.51	15.8	0.479	0.838	5.86	26.3	0.479	0.838	82.9	36.4	38.1

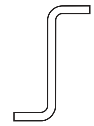
Notes:

1. Shear and moment strengths given are nominal strengths. To obtain available strengths, these values must be modified by safety factors (ASD) or resistance factors (LRFD).
2. M_{web}, M_{flange} and M_{lip} are the highest nominal moments at which the web, flange and lip, respectively, are fully effective.
3. The distortional buckling limit state is not considered in this table. Consideration of distortional buckling may result in lower strengths when restraint against distortional buckling is not provided. See Table II-9 for distortional buckling strengths.

Table II - 5

Beam Properties
Z-Sections Without Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD)
Ω_v = 1.60 (ASD)
ϕ_v = 0.95 (LRFD)



Section	F_y = 33 ksi				F_y = 50 ksi				Maximum Fully-Effective Moment [2]	
	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	V_n [1] kips	M_{nxo} [1] kip-in.	S_e in.^{3}	I_e in.^{4}	M_{web} kip-in.	M_{flange} kip-in.
8ZU1.25x105	15.1	65.0	1.97	7.88	18.6	98.5	1.97	7.88	124	123
8ZU1.25x090	11.1	56.2	1.70	6.82	13.6	83.6	1.67	6.82	77.4	75.9
8ZU1.25x075	7.70	46.7	1.41	5.73	8.04	64.5	1.29	5.73	44.6	43.0
8ZU1.25x060	4.10	33.9	1.03	4.63	4.10	46.0	0.919	4.49	22.7	21.5
8ZU1.25x048	2.09	24.1	0.730	3.59	2.09	32.2	0.644	3.40	11.6	10.9
6ZU1.25x105	11.3	41.6	1.26	3.78	17.1	63.1	1.26	3.78	137	78.8
6ZU1.25x090	9.70	36.1	1.09	3.28	13.6	54.1	1.08	3.28	85.3	48.7
6ZU1.25x075	7.70	30.1	0.914	2.77	9.47	43.8	0.876	2.77	48.9	27.6
6ZU1.25x060	4.92	23.2	0.704	2.24	5.59	32.3	0.646	2.18	24.9	13.9
6ZU1.25x048	2.85	17.0	0.514	1.76	2.85	23.0	0.460	1.70	12.6	7.00
4ZU1.25x090	6.14	19.9	0.603	1.21	9.30	29.8	0.595	1.21	212	26.8
4ZU1.25x075	5.16	16.6	0.505	1.02	7.82	23.9	0.479	1.02	121	15.3
4ZU1.25x060	4.16	12.7	0.386	0.829	6.06	18.2	0.364	0.803	60.9	7.71
4ZU1.25x048	3.15	9.65	0.292	0.647	3.88	13.8	0.276	0.624	30.8	3.90
4ZU1.25x036	1.77	6.75	0.204	0.465	1.87	9.71	0.194	0.447	12.8	1.63
3.625ZU1.25x090	5.47	17.3	0.524	0.950	8.29	25.8	0.517	0.950	212	23.3
3.625ZU1.25x075	4.60	14.5	0.439	0.805	6.98	20.8	0.416	0.805	121	13.3
3.625ZU1.25x060	3.72	11.0	0.335	0.655	5.63	15.8	0.315	0.634	60.8	6.72
3.625ZU1.25x048	3.00	8.36	0.253	0.511	3.88	11.9	0.239	0.492	30.7	3.40
3.625ZU1.25x036	1.77	5.83	0.176	0.367	2.09	8.35	0.167	0.352	12.8	1.42
2.5ZU1.25x090	3.47	10.4	0.314	0.392	5.25	15.4	0.309	0.392	213	14.0
2.5ZU1.25x075	2.93	8.70	0.264	0.334	4.44	12.4	0.247	0.334	120	8.01
2.5ZU1.25x060	2.38	6.59	0.200	0.273	3.61	9.31	0.186	0.264	60.2	4.06
2.5ZU1.25x048	1.93	4.95	0.150	0.213	2.92	6.98	0.140	0.204	30.3	2.07
2.5ZU1.25x036	1.46	3.40	0.103	0.152	2.18	4.81	0.0963	0.145	12.5	0.866
1.5ZU1.25x090	1.68	5.24	0.159	0.119	2.55	7.78	0.156	0.119	217	7.06
1.5ZU1.25x075	1.45	4.43	0.134	0.102	2.19	6.22	0.124	0.102	121	4.09
1.5ZU1.25x060	1.19	3.35	0.101	0.0845	1.81	4.66	0.0931	0.0814	59.9	2.10
1.5ZU1.25x048	0.978	2.49	0.0754	0.0664	1.48	3.45	0.0690	0.0631	29.9	1.07
1.5ZU1.25x036	0.751	1.68	0.0510	0.0473	1.14	2.34	0.0468	0.0447	12.3	0.454

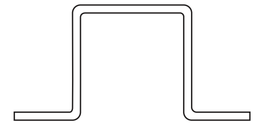
Notes:

1. Shear and moment strengths given are nominal strengths. To obtain available strengths, these values must be modified by safety factors (ASD) or resistance factors (LRFD).
2. M_{web} and M_{flange} are the highest nominal moments at which the web and flange, respectively, are fully effective.

Table II - 6

Beam Properties
Hat-Sections Without Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD)
Ω_v = 1.60 (ASD)
ϕ_v = 0.95 (LRFD)



Section	F_y = 33 ksi						F_y = 50 ksi					
		Compression on Top			Compression on Bottom			Compression on Top			Compression on Bottom	
	V_n [1] kips	M_{nyo} [1,2] kip-in.	S_e in.^{3}	I_e in.^{4}	M_{nyo} [1,2] kip-in.	S_e in.^{3}	V_n [1] kips	M_{nyo} [1,2] kip-in.	S_e in.^{3}	I_e in.^{4}	M_{nyo} [1,2] kip-in.	S_e in.^{3}
10HU5x075	12.7	137	4.15	23.5	124	3.77	12.7	203	4.06	22.6	173	3.46
8HU12x135	39.3	222	6.72	34.4	234	7.09	59.6	326	6.52	32.4	355	7.09
8HU12x105	30.2	155	4.70	23.8	169	5.13	37.1	228	4.55	22.3	243	4.87
8HU8x105	30.2	153	4.63	22.6	160	4.83	37.1	225	4.51	21.5	236	4.72
8HU8x075	15.4	95.5	2.89	14.0	96	2.90	16.1	141	2.81	13.2	134	2.68
8HU4x075	15.4	92.5	2.80	12.4	90.0	2.73	16.1	137	2.75	12.3	128	2.55
8HU4x060	8.20	69.8	2.12	9.59	63.7	1.93	8.20	104	2.07	9.25	89	1.79
6HU9x135	28.6	143	4.33	16.4	146	4.43	43.4	212	4.23	15.8	222	4.43
6HU9x105	22.5	99.4	3.01	11.55	104	3.16	34.1	146	2.93	10.89	158	3.16
6HU6x105	22.5	97.3	2.95	10.42	98.4	2.98	34.1	144	2.88	10.27	149	2.98
6HU6x075	15.4	58.8	1.78	6.58	61.5	1.86	18.9	86.7	1.73	6.25	89.9	1.80
6HU3x075	15.4	55.1	1.67	5.36	55.1	1.67	18.9	83.4	1.67	5.36	83.4	1.67
6HU3x060	9.85	41.8	1.27	4.15	41.4	1.25	11.2	62.3	1.25	4.15	59.2	1.18
6HU3x048	5.70	31.7	0.962	3.25	29.7	0.901	5.70	47.1	0.943	3.17	41.9	0.838
4HU6x135	17.9	77.3	2.34	5.42	77.3	2.34	27.2	116	2.33	5.42	117	2.34
4HU6x105	14.2	53.9	1.63	3.96	54.3	1.64	21.5	80.0	1.60	3.93	82.2	1.64
4HU4x105	14.2	51.2	1.55	3.39	51.2	1.55	21.5	77.6	1.55	3.39	77.6	1.55
4HU4x075	10.3	30.8	0.935	2.18	30.9	0.937	15.6	45.8	0.917	2.18	46.8	0.937
4HU2x075	10.3	27.7	0.839	1.70	27.7	0.839	15.6	41.9	0.839	1.70	41.9	0.839
4HU2x060	8.33	20.6	0.623	1.30	20.6	0.623	12.1	31.2	0.623	1.30	31.2	0.623
4HU2x048	6.30	15.4	0.468	1.002	15.4	0.468	7.76	23.4	0.468	1.002	23.4	0.468
3HU4.5x135	12.6	50.0	1.52	2.47	50.0	1.52	19.1	75.8	1.52	2.47	75.8	1.52
3HU4.5x105	10.0	34.7	1.05	1.80	34.7	1.05	15.2	52.5	1.05	1.80	52.6	1.05
3HU3x105	10.0	32.7	0.992	1.53	32.7	0.992	15.2	49.6	0.992	1.53	49.6	0.992
3HU3x075	7.35	19.3	0.585	0.977	19.3	0.585	11.1	29.3	0.585	0.977	29.3	0.585

Notes:

1. Shear and moment strengths given are nominal strengths. To obtain available strengths, these values must be modified by safety factors (ASD) or resistance factors (LRFD).
2. Y-axis is horizontal.

1.3 Distortional Buckling Flexural Strength Tables

Tables II-7, II-8 and II-9 provide computed distortional buckling properties for the representative C-shapes, SSMA studs and Z-shapes with lips, respectively. The values in these tables have been calculated for use with Section C3.1.4(b).

(a) The values in the column under the heading L_{cr} can be used to calculate β with *Eq.* C3.1.4-11. β accounts for the moment gradient over the distortional buckling length, L_m. It is always conservative to take β as 1.0. In cases where the bending moment is almost constant over the length L_m, β will be calculated to be approximately 1.0.

(b) The values in the column under the heading F_d/β may be multiplied by β to give F_d directly, in lieu of using *Eq.* C3.1.4-10. This will result in the neglect of any rotational stiffness provided by bracing or sheathing, which is conservative where such rotational stiffness exists. These values have been calculated assuming that the unbraced length for distortional buckling, L_m, is greater than or equal to L_{cr}. In cases where the unbraced length is shorter than L_{cr}, these values will produce conservative strengths.

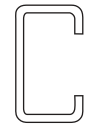
(c) Where a known rotational stiffness, k_ϕ, from bracing or sheathing is available, the values in the columns under the headings $k_{\phi fe}$, $\tilde{k}_{\phi fg}$, $k_{\phi we}$ and $\tilde{k}_{\phi wg}$ may be used in *Eq.* C3.1.4-10 to calculate a more exact value of F_d. These values have been calculated assuming that the unbraced length for distortional buckling, L_m, is greater than or equal to L_{cr}. In cases where the unbraced length is shorter than L_{cr}, these values will produce conservative strengths.

(d) The values in the columns under the headings $M_{n(\beta=1)}$ are nominal distortional buckling strengths calculated assuming β=1.0, i.e. no moment gradient. These values will be conservative if β>1.0.

Table II - 7

Distortional Buckling Properties
Flexural Strength
C-Sections With Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD)

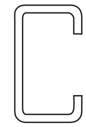


Section	Per Section 3.1.4(b)							
	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d/β ksi	M_n[1]$_{(\beta=1)}$ (F_y=33 ksi) kip-in.	M_n[1]$_{(\beta=1)}$ (F_y=55 ksi) kip-in.
12CS4x105	29.5	1.01	0.0393	0.919	0.00576	42.8	223	309
12CS4x085	31.8	0.520	0.0272	0.477	0.00405	31.9	163	224
12CS4x070	34.2	0.282	0.0193	0.262	0.00290	24.5	122	166
12CS3.5x105	27.3	1.02	0.0326	0.943	0.00669	50.0	216	302
12CS3.5x085	29.4	0.524	0.0225	0.488	0.00470	37.2	159	220
12CS3.5x070	31.6	0.284	0.0159	0.267	0.00337	28.6	120	163
12CS2.5x105	22.3	1.05	0.0213	1.03	0.00974	67.0	195	278
12CS2.5x085	24.1	0.538	0.0146	0.526	0.00684	49.6	145	203
12CS2.5x070	25.9	0.291	0.0102	0.285	0.00490	38.0	110	152
10CS4x105	28.2	1.19	0.0431	1.06	0.00370	48.1	182	254
10CS4x085	30.4	0.613	0.0299	0.555	0.00260	36.0	134	185
10CS4x070	32.7	0.334	0.0211	0.305	0.00186	27.8	101	138
10CS4x065	33.6	0.265	0.0185	0.243	0.00163	25.3	90.4	123
10CS3.5x105	26.1	1.20	0.0358	1.08	0.00431	57.1	175	247
10CS3.5x085	28.1	0.618	0.0247	0.565	0.00302	42.7	130	181
10CS3.5x070	30.2	0.336	0.0174	0.310	0.00216	33.0	98.5	135
10CS3.5x065	31.1	0.266	0.0153	0.247	0.00190	29.9	88.4	121
10CS2.5x105	21.3	1.24	0.0234	1.16	0.00631	80.8	154	228
10CS2.5x085	23.0	0.634	0.0160	0.598	0.00442	60.4	119	168
10CS2.5x070	24.8	0.343	0.0112	0.326	0.00316	46.6	90.8	127
10CS2.5x065	25.5	0.272	0.00979	0.259	0.00278	42.3	81.8	113
10CS2x105	18.6	1.27	0.0185	1.23	0.00812	93.9	137	212
10CS2x085	20.1	0.647	0.0125	0.630	0.00569	70.1	110	158
10CS2x070	21.7	0.349	0.00873	0.341	0.00407	54.0	84.7	119
10CS2x065	22.3	0.276	0.00760	0.270	0.00357	48.9	76.4	107
9CS2.5x105	20.7	1.36	0.0247	1.25	0.00490	88.4	133	202
9CS2.5x085	22.4	0.697	0.0169	0.648	0.00343	66.3	105	150
9CS2.5x070	24.1	0.378	0.0118	0.354	0.00245	51.3	80.9	113
9CS2.5x065	24.8	0.299	0.0103	0.282	0.00215	46.6	72.9	102
9CS2.5x059	25.8	0.221	0.00863	0.209	0.00181	41.2	63.6	88.2
8CS4x105	26.7	1.46	0.0483	1.28	0.00215	54.4	141	198
8CS4x085	28.7	0.752	0.0334	0.673	0.00151	40.8	105	145
8CS4x070	30.9	0.410	0.0236	0.372	0.00108	31.7	79.3	109
8CS4x065	31.8	0.325	0.0207	0.297	0.000944	28.8	71.2	97.3
8CS4x059	33.0	0.241	0.0173	0.221	0.000796	25.4	61.8	84.1
8CS3.5x105	24.6	1.47	0.0400	1.30	0.00250	65.2	136	193
8CS3.5x085	26.6	0.758	0.0276	0.682	0.00175	49.0	101	142
8CS3.5x070	28.6	0.413	0.0195	0.376	0.00125	38.0	77.2	107
8CS3.5x065	29.4	0.327	0.0171	0.300	0.00110	34.5	69.4	95.5
8CS3.5x059	30.6	0.242	0.0143	0.223	0.000926	30.5	60.4	82.7
8CS2.5x105	20.1	1.51	0.0262	1.37	0.00369	96.5	113	176
8CS2.5x085	21.7	0.776	0.0179	0.712	0.00258	72.6	91.7	131
8CS2.5x070	23.4	0.421	0.0126	0.390	0.00184	56.4	70.8	99.7
8CS2.5x065	24.1	0.334	0.0109	0.311	0.00161	51.3	63.9	89.6
8CS2.5x059	25.1	0.246	0.00916	0.230	0.00136	45.4	55.9	77.8

Table II - 7

Distortional Buckling Properties
Flexural Strength
C-Sections With Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD)



Section	Per Section 3.1.4(b)							
	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d/β ksi	M_n[1] $_{(\beta=1)}$ (F_y=33 ksi) kip-in.	M_n[1] $_{(\beta=1)}$ (F_y=55 ksi) kip-in.
8CS2x105	17.6	1.55	0.0207	1.43	0.00477	117	99.1	164
8CS2x085	19.0	0.791	0.0140	0.740	0.00333	88.2	80.8	123
8CS2x070	20.5	0.428	0.00977	0.404	0.00238	68.5	65.8	93.9
8CS2x065	21.1	0.339	0.00851	0.321	0.00208	62.3	59.7	84.5
8CS2x059	21.9	0.250	0.00709	0.237	0.00176	55.1	52.3	73.6
7CS4x105	25.8	1.65	0.0516	1.44	0.00155	58.1	121	171
7CS4x085	27.8	0.850	0.0357	0.758	0.00108	43.7	90.4	126
7CS4x070	29.9	0.464	0.0253	0.420	0.000774	34.0	68.7	94.4
7CS4x065	30.8	0.368	0.0221	0.335	0.000679	30.9	61.7	84.6
7CS4x059	31.9	0.273	0.0185	0.250	0.000572	27.3	53.7	73.2
7CS2.5x105	19.5	1.70	0.0280	1.52	0.00267	105	93.7	150
7CS2.5x085	21.0	0.876	0.0191	0.794	0.00186	79.5	76.5	113
7CS2.5x070	22.6	0.476	0.0134	0.437	0.00133	61.9	60.6	85.9
7CS2.5x065	23.3	0.377	0.0117	0.348	0.00116	56.4	54.9	77.3
7CS2.5x059	24.2	0.279	0.00979	0.259	0.000980	49.9	48.1	67.3
6CS4x105	24.8	1.89	0.0557	1.66	0.00106	62.6	102	144
6CS4x085	26.7	0.980	0.0386	0.873	0.000741	47.1	76.3	106
6CS4x070	28.7	0.536	0.0273	0.484	0.000528	36.7	58.2	80.2
6CS4x065	29.6	0.426	0.0239	0.387	0.000464	33.4	52.4	72.0
6CS4x059	30.7	0.315	0.0200	0.288	0.000391	29.6	45.6	62.4
6CS2.5x105	18.7	1.96	0.0303	1.73	0.00183	115	76.0	125
6CS2.5x085	20.2	1.01	0.0207	0.907	0.00128	87.2	62.1	94.3
6CS2.5x070	21.8	0.549	0.0145	0.500	0.000909	68.1	50.6	72.2
6CS2.5x065	22.4	0.435	0.0126	0.399	0.000797	62.0	45.9	65.1
6CS2.5x059	23.3	0.322	0.0106	0.297	0.000671	55.0	40.4	56.8
4CS4x105	22.4	2.75	0.0683	2.42	0.000388	75.3	63.8	92.5
4CS4x085	24.1	1.43	0.0472	1.28	0.000271	57.0	49.0	69.0
4CS4x070	26.0	0.784	0.0334	0.712	0.000193	44.5	37.7	52.5
4CS4x065	26.7	0.623	0.0292	0.569	0.000169	40.6	34.1	47.2
4CS4x059	27.8	0.462	0.0245	0.425	0.000143	36.0	29.8	41.1
4CS2.5x105	16.9	2.83	0.0371	2.48	0.000676	141	44.1	73.5
4CS2.5x085	18.3	1.46	0.0253	1.31	0.000471	107	36.3	58.6
4CS2.5x070	19.7	0.800	0.0178	0.725	0.000334	84.3	30.3	45.4
4CS2.5x065	20.3	0.636	0.0155	0.579	0.000293	76.9	28.2	41.1
4CS2.5x059	21.1	0.471	0.0130	0.432	0.000246	68.4	25.3	36.1
4CS2x105	15.8	2.83	0.0284	2.50	0.000770	183	37.9	63.2
4CS2x085	17.2	1.46	0.0191	1.32	0.000531	142	31.3	52.1
4CS2x070	18.6	0.798	0.0132	0.729	0.000374	113	26.1	42.6
4CS2x065	19.2	0.634	0.0114	0.582	0.000328	103	24.3	38.8
4CS2x059	20.0	0.469	0.00951	0.434	0.000274	92.3	22.2	34.3

Notes:

1. Flexural strengths given are nominal strengths. To obtain available strengths, these values must be modified by the safety factor (ASD) or resistance factor (LRFD).

Table II - 8

Distortional Buckling Properties
SSMA Studs – Flexural Strength
C-Sections With Lips

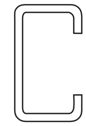
Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD)

Section	Per Section 3.1.4(b)							
	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d/β ksi	M_n[1]$_{(\beta=1)}$ (F_y=33 ksi) kip-in.	M_n[1]$_{(\beta=1)}$ (F_y=50 ksi) kip-in.
1200S250-97	17.9	1.05	0.0256	1.06	0.0142	53.2	171	226
1200S250-68	21.7	0.328	0.0128	0.326	0.00698	33.1	105	136
1200S250-54*	24.5	0.156	0.00811	0.154	0.00441	24.7	75.9	97.5
1200S200-97	15.7	1.07	0.0189	1.18	0.0180	60.9	159	212
1200S200-68	19.0	0.334	0.00953	0.352	0.00889	37.2	98.2	128
1200S200-54*	21.5	0.158	0.00606	0.164	0.00562	27.5	70.9	91.4
1200S162-97	12.1	1.15	0.0162	1.54	0.0292	59.1	140	186
1200S162-68	14.5	0.359	0.00845	0.435	0.0146	34.5	85.1	110
1200S162-54*	16.4	0.169	0.00543	0.196	0.00925	24.9	60.7	78.1
1000S250-97	17.0	1.24	0.0283	1.17	0.00924	64.3	139	186
1000S250-68	20.7	0.387	0.0141	0.368	0.00453	40.6	87.2	114
1000S250-54	23.4	0.184	0.00889	0.176	0.00285	30.6	63.5	82.1
1000S250-43*	26.4	0.0891	0.00566	0.0857	0.00180	23.4	46.2	–
1000S200-97	14.9	1.26	0.0209	1.28	0.0118	77.7	128	175
1000S200-68	18.1	0.394	0.0105	0.391	0.00579	48.2	81.9	108
1000S200-54	20.5	0.187	0.00665	0.184	0.00364	36.0	59.9	77.8
1000S200-43*	23.2	0.0902	0.00424	0.0891	0.00231	27.4	43.7	–
1000S162-97	11.4	1.38	0.0183	1.61	0.0193	79.5	112	155
1000S162-68	13.7	0.427	0.00938	0.467	0.00957	47.1	71.7	94.1
1000S162-54	15.6	0.200	0.00600	0.214	0.00606	34.3	51.9	67.4
1000S162-43*	17.6	0.0960	0.00384	0.100	0.00386	25.5	37.6	–
800S250-97	16.1	1.51	0.0317	1.35	0.00544	76.9	106	144
800S250-68	19.5	0.472	0.0158	0.433	0.00265	49.2	68.2	89.8
800S250-54	22.1	0.225	0.00996	0.209	0.00166	37.4	50.2	65.3
800S250-43	25.0	0.110	0.00633	0.103	0.00105	28.8	36.8	–
800S200-97	14.0	1.54	0.0236	1.44	0.00699	97.6	92.4	136
800S200-68	17.1	0.481	0.0118	0.453	0.00340	61.6	64.2	85.4
800S200-54	19.4	0.229	0.00745	0.217	0.00213	46.5	47.5	62.4
800S200-43	21.9	0.111	0.00475	0.106	0.00135	35.6	35.0	–
800S200-33*	25.2	0.0481	0.00280	0.0464	0.000790	26.3	24.3	–
800S162-97	10.6	1.70	0.0209	1.74	0.0116	106	80.1	120
800S162-68	12.9	0.522	0.0106	0.521	0.00569	64.1	56.5	75.3
800S162-54	14.7	0.245	0.00674	0.243	0.00359	47.3	41.7	54.8
800S162-43	16.6	0.118	0.00431	0.116	0.00227	35.5	30.6	–
800S162-33*	19.1	0.0505	0.00255	0.0498	0.00134	25.8	21.1	–
800S137-97	8.04	1.89	0.0196	2.31	0.0195	107	70.9	107
800S137-68	9.66	0.581	0.0103	0.653	0.00970	61.6	49.7	66.1
800S137-54	10.9	0.271	0.00667	0.293	0.00615	44.0	36.3	47.5
800S137-43	12.3	0.128	0.00430	0.136	0.00393	32.1	26.3	–
800S137-33*	14.2	0.0543	0.00255	0.0564	0.00232	22.7	17.9	–
600S250-97	14.9	1.94	0.0367	1.66	0.00273	91.3	71.5	103
600S250-68	18.2	0.612	0.0182	0.546	0.00132	59.3	49.2	65.2
600S250-54	20.6	0.293	0.0115	0.267	0.000821	45.4	36.6	48.0
600S250-43	23.2	0.143	0.00731	0.133	0.000517	35.2	27.1	–
600S200-97	13.0	1.98	0.0273	1.74	0.00353	121	61.7	93.5
600S200-68	15.9	0.623	0.0136	0.564	0.00170	77.5	45.1	61.8
600S200-54	18.0	0.297	0.00862	0.274	0.00106	59.0	34.5	45.7
600S200-43	20.4	0.145	0.00549	0.135	0.000668	45.5	25.7	–
600S200-33	23.4	0.0631	0.00324	0.0598	0.000390	33.9	18.0	–

Table II - 8

Distortional Buckling Properties
SSMA Studs – Flexural Strength
C-Sections With Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD)



Section	Per Section 3.1.4(b)							
	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d/β ksi	$M_n{}^1{}_{(\beta=1)}$ (F_y=33 ksi) kip-in.	$M_n{}^1{}_{(\beta=1)}$ (F_y=50 ksi) kip-in.
600S162-97	9.84	2.19	0.0244	2.00	0.00594	138	52.8	80.0
600S162-68	12.0	0.674	0.0123	0.621	0.00288	85.3	38.8	54.7
600S162-54	13.6	0.318	0.00781	0.296	0.00180	63.8	30.4	40.5
600S162-43	15.4	0.153	0.00498	0.144	0.00114	48.5	22.6	–
600S162-33	17.8	0.0659	0.00294	0.0627	0.000665	35.6	15.8	–
600S137-97	7.34	2.50	0.0236	2.50	0.0101	148	46.1	69.8
600S137-68	8.91	0.755	0.0122	0.735	0.00500	86.8	34.0	48.3
600S137-54	10.1	0.351	0.00779	0.339	0.00315	63.0	26.6	35.5
600S137-43	11.4	0.166	0.00499	0.161	0.00199	46.8	19.7	–
600S137-33	13.2	0.0706	0.00296	0.0683	0.00117	33.6	13.7	–
550S162-68	11.7	0.728	0.0129	0.660	0.00234	91.3	34.3	49.4
550S162-54	13.3	0.343	0.00816	0.316	0.00146	68.5	27.4	36.7
550S162-43	15.1	0.166	0.00521	0.154	0.000921	52.2	20.6	–
550S162-33	17.4	0.0715	0.00308	0.0675	0.000538	38.5	14.4	–
400S200-68	14.4	0.898	0.0167	0.794	0.000632	97.8	26.2	38.5
400S200-54	16.3	0.432	0.0106	0.391	0.000392	75.1	21.3	28.9
400S200-43	18.4	0.212	0.00672	0.195	0.000246	58.3	16.3	–
400S200-33	21.2	0.0927	0.00397	0.0871	0.000143	43.8	11.5	–
400S162-68	10.8	0.964	0.0151	0.841	0.00109	111	22.2	33.7
400S162-54	12.3	0.458	0.00958	0.408	0.000675	84.4	18.1	25.5
400S162-43	13.9	0.222	0.00611	0.202	0.000423	64.9	14.3	–
400S162-33	16.1	0.0962	0.00361	0.0894	0.000246	48.2	10.1	–
400S137-68	8.01	1.07	0.0151	0.932	0.00192	118	19.2	29.1
400S137-54	9.11	0.501	0.00960	0.443	0.00120	87.4	15.7	22.3
400S137-43	10.3	0.239	0.00614	0.215	0.000754	66.0	12.5	–
400S137-33	11.9	0.102	0.00363	0.0939	0.000439	48.2	8.83	–
362S200-68	14.0	0.983	0.0175	0.867	0.000496	103	23.0	34.3
362S200-54	15.9	0.473	0.0111	0.428	0.000307	79.0	18.8	25.8
362S200-43	18.0	0.232	0.00706	0.214	0.000193	61.5	14.5	–
362S200-33	20.7	0.102	0.00417	0.0956	0.000112	46.2	10.3	–
362S162-68	10.6	1.05	0.0159	0.911	0.000855	117	19.5	29.5
362S162-54	12.0	0.500	0.0101	0.444	0.000530	89.1	15.9	22.7
362S162-43	13.6	0.243	0.00642	0.220	0.000332	68.6	12.7	–
362S162-33	15.7	0.106	0.00379	0.0978	0.000193	51.1	9.07	–
362S137-68	7.81	1.17	0.0158	0.997	0.00152	125	16.8	25.5
362S137-54	8.88	0.546	0.0101	0.477	0.000946	92.7	13.8	19.9
362S137-43	10.1	0.261	0.00645	0.233	0.000593	70.2	11.1	–
362S137-33	11.6	0.112	0.00381	0.102	0.000345	51.4	7.90	–
350S162-68	10.5	1.09	0.0162	0.938	0.000785	120	18.6	28.1
350S162-54	11.9	0.516	0.0102	0.458	0.000487	90.8	15.2	21.8
350S162-43	13.5	0.251	0.00653	0.227	0.000305	70.0	12.2	–
350S162-33	15.5	0.109	0.00386	0.101	0.000177	52.1	8.71	–
250S162-68	9.63	1.47	0.0191	1.26	0.000344	140	11.9	18.0
250S162-54	10.9	0.702	0.0121	0.620	0.000213	107	9.76	14.7
250S162-43	12.4	0.343	0.00773	0.310	0.000133	83.0	7.97	–
250S162-33	14.3	0.150	0.00456	0.139	0.0000769	62.2	5.95	–
250S137-68	7.11	1.61	0.0191	1.33	0.000619	149	10.2	15.4
250S137-54	8.09	0.759	0.0122	0.647	0.000383	112	8.40	12.7
250S137-43	9.17	0.365	0.00777	0.321	0.000239	85.6	6.88	–
250S137-33	10.6	0.158	0.00459	0.142	0.000138	63.3	5.17	–

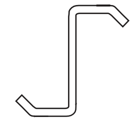
Notes:

1. Flexural strengths given are nominal strengths. To obtain available strengths, these values must be modified by the safety factor (ASD) or resistance factor(LRFD).

Table II - 9

Distortional Buckling Properties
Flexural Strength
Z-Sections With Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD)

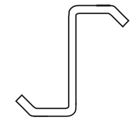


Section	Per Section 3.1.4(b)							
	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d/β ksi	$M_n{}^1{}_{(\beta=1)}$ (F_y=33 ksi) kip-in.	$M_n{}^1{}_{(\beta=1)}$ (F_y=55 ksi) kip-in.
12ZS3.25x105	24.3	1.07	0.0391	0.987	0.00832	43.5	206	287
12ZS3.25x085	26.6	0.544	0.0263	0.505	0.00566	32.8	152	209
12ZS3.25x070	28.9	0.293	0.0183	0.274	0.00400	25.4	114	155
12ZS2.75x105	21.9	1.09	0.0326	1.04	0.0101	49.9	197	276
12ZS2.75x085	24.0	0.553	0.0219	0.526	0.00687	37.5	146	201
12ZS2.75x070	26.1	0.297	0.0152	0.284	0.00485	29.0	110	150
12ZS2.25x105	19.3	1.12	0.0268	1.11	0.0127	56.5	186	262
12ZS2.25x085	21.2	0.566	0.0180	0.559	0.00866	42.2	138	191
12ZS2.25x070	23.0	0.303	0.0124	0.299	0.00612	32.5	104	142
10ZS3.25x105	23.2	1.26	0.0429	1.12	0.00537	49.4	168	235
10ZS3.25x085	25.4	0.640	0.0288	0.579	0.00365	37.5	125	172
10ZS3.25x070	27.6	0.345	0.0200	0.317	0.00257	29.3	94.1	129
10ZS3.25x065	28.5	0.273	0.0174	0.252	0.00224	26.7	84.5	115
10ZS3.25x059	29.8	0.201	0.0145	0.186	0.00187	23.7	73.4	99.7
10ZS2.75x105	20.9	1.28	0.0358	1.17	0.00654	58.0	160	226
10ZS2.75x085	22.9	0.651	0.0240	0.599	0.00444	43.9	119	166
10ZS2.75x070	24.9	0.350	0.0166	0.326	0.00313	34.2	90.6	125
10ZS2.75x065	25.7	0.277	0.0145	0.258	0.00273	31.1	81.4	112
10ZS2.75x059	26.9	0.204	0.0120	0.191	0.00228	27.6	70.8	96.6
10ZS2.25x105	18.4	1.32	0.0295	1.24	0.00827	67.6	151	215
10ZS2.25x085	20.2	0.666	0.0198	0.628	0.00562	51.0	113	158
10ZS2.25x070	22.0	0.357	0.0137	0.339	0.00396	39.6	85.8	119
10ZS2.25x065	22.7	0.282	0.0119	0.269	0.00346	36.0	77.2	106
10ZS2.25x059	23.7	0.207	0.00983	0.198	0.00289	31.9	67.2	92.1
9ZS2.25x105	17.9	1.45	0.0312	1.33	0.00644	73.7	132	191
9ZS2.25x085	19.7	0.731	0.0209	0.677	0.00437	55.8	100	141
9ZS2.25x070	21.4	0.393	0.0144	0.367	0.00307	43.5	76.5	106
9ZS2.25x065	22.1	0.311	0.0125	0.291	0.00268	39.6	68.9	95.3
9ZS2.25x059	23.1	0.228	0.0104	0.215	0.00224	35.1	60.1	82.7
8ZS3.25x105	21.9	1.54	0.0480	1.34	0.00313	56.2	130	183
8ZS3.25x085	24.1	0.782	0.0323	0.695	0.00212	43.0	97.0	135
8ZS3.25x070	26.1	0.423	0.0224	0.382	0.00149	33.7	73.8	101
8ZS3.25x065	27.0	0.335	0.0195	0.304	0.00130	30.7	66.4	90.9
8ZS3.25x059	28.2	0.247	0.0162	0.226	0.00108	27.4	57.8	78.8
8ZS2.75x105	19.7	1.56	0.0401	1.38	0.00382	67.0	123	175
8ZS2.75x085	21.7	0.795	0.0269	0.712	0.00259	51.1	92.7	130
8ZS2.75x070	23.6	0.429	0.0186	0.390	0.00182	40.0	70.8	98.0
8ZS2.75x065	24.3	0.339	0.0162	0.310	0.00159	36.5	63.8	87.9
8ZS2.75x059	25.4	0.250	0.0134	0.230	0.00132	32.5	55.6	76.3
8ZS2.25x105	17.4	1.60	0.0331	1.44	0.00486	80.1	112	166
8ZS2.25x085	19.1	0.812	0.0221	0.738	0.00329	61.0	87.2	123
8ZS2.25x070	20.8	0.437	0.0153	0.402	0.00231	47.7	67.0	93.5
8ZS2.25x065	21.5	0.346	0.0133	0.319	0.00202	43.5	60.4	84.0
8ZS2.25x059	22.4	0.254	0.0110	0.236	0.00168	38.6	52.8	73.0
7ZS2.25x105	16.8	1.80	0.0354	1.59	0.00352	87.1	93.5	142
7ZS2.25x085	18.5	0.915	0.0237	0.819	0.00238	66.5	74.5	106
7ZS2.25x070	20.1	0.493	0.0164	0.448	0.00167	52.2	57.4	80.6
7ZS2.25x065	20.8	0.390	0.0142	0.356	0.00146	47.7	51.9	72.5
7ZS2.25x059	21.7	0.287	0.0118	0.264	0.00121	42.4	45.5	63.1

Table II - 9

Distortional Buckling Properties
Flexural Strength
Z-Sections With Lips

Ω_b = 1.67 (ASD)
ϕ_b = 0.90 (LRFD)



Section	Per Section 3.1.4(b)							
	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d/β ksi	M_n[1] $(\beta=1)$ (F_y=33 ksi) kip-in.	M_n[1] $(\beta=1)$ (F_y=55 ksi) kip-in.
6ZS2.25x105	16.2	2.07	0.0383	1.79	0.00243	94.8	75.8	118
6ZS2.25x085	17.8	1.05	0.0256	0.929	0.00164	72.7	61.9	88.6
6ZS2.25x070	19.3	0.568	0.0177	0.510	0.00115	57.3	48.0	67.7
6ZS2.25x065	20.0	0.449	0.0153	0.406	0.001000	52.4	43.5	61.0
6ZS2.25x059	20.9	0.331	0.0127	0.302	0.000832	46.7	38.2	53.2
4ZS2.25x070	17.5	0.823	0.0217	0.733	0.000423	70.5	29.8	42.6
4ZS2.25x065	18.0	0.653	0.0188	0.585	0.000368	64.6	27.1	38.5
4ZS2.25x059	18.9	0.482	0.0156	0.436	0.000306	57.8	23.9	33.7
3.5ZS1.5x070	10.6	1.03	0.0180	0.885	0.000750	102	18.6	29.5
3.5ZS1.5x065	11.0	0.811	0.0157	0.704	0.000656	92.8	17.3	26.8
3.5ZS1.5x059	11.4	0.596	0.0130	0.522	0.000548	82.3	15.8	23.5

Notes:

1. Flexural strengths given are nominal strengths. To obtain available strengths, these values must be modified by the safety factor (ASD) or resistance factor (LRFD).

1.4 Calculation of L_u

For members bent about the centroidal axis perpendicular to the web, calculation of lateral buckling strength is unnecessary when the unbraced length is less than a length, L_u, which results in a critical elastic flexural stress, F_e, that is $2.78F_y$. L_u may be calculated using the following formulae. All terms are as defined in Section C3.1.2 of the *Specification.*

(a) For singly-, doubly-, and point symmetric sections:

$$L_u = \left\{ \frac{GJ}{2C_1} + \left[\frac{C_2}{C_1} + \left(\frac{GJ}{2C_1} \right)^2 \right]^{0.5} \right\}^{0.5}$$

where

(1) For singly- and doubly-symmetric sections:

$$C_1 = \frac{7.72}{AE}\left[\frac{K_y F_y S_f}{C_b \pi r_y} \right]^2$$

$$C_2 = \frac{\pi^2 E C_w}{(K_t)^2}$$

(2) For point-symmetric sections:

$$C_1 = \frac{30.9}{AE}\left[\frac{K_y F_y S_f}{C_b \pi r_y} \right]^2$$

$$C_2 = \frac{\pi^2 E C_w}{(K_t)^2}$$

(b) For I-, C- or Z-sections bent about the centroidal axis perpendicular to the web (x-axis), in lieu of (a), the following equations may be used:

(1) For doubly-symmetric I-sections and singly-symmetric C-sections:

$$L_u = \sqrt{\frac{0.36 C_b \pi^2 E d I_{yc}}{F_y S_f (K_y)^2}}$$

(2) For point-symmetric Z-sections:

$$L_u = \sqrt{\frac{0.18 C_b \pi^2 E d I_{yc}}{F_y S_f (K_y)^2}}$$

(c) For closed box members:

$$L_u = \frac{0.36 C_b \pi}{F_y S_f}\sqrt{EGJI_y} \qquad (Eq.\ C3.1.2.2\text{-}1)$$

1.5 Notes on the Charts

(a) With the exception of the SSMA studs, the specific sections listed in these charts are not necessarily stock sections. They are included primarily as a guide in the design of cold-formed steel structural members.

(b) The section designations listed in these charts are a subset of those for which dimensions and properties are given in Tables I-1, I-2 and I-4. This subset is intended to represent those sections most commonly used in routine design.

(c) The nominal flexural strength, M_n, is given as a function of unbraced length. In these charts the torsional unbraced length is assumed to equal the y-axis unbraced length and $K_y = K_t = 1.0$.

(d) The effects of standard factory punchouts in SSMA studs have been included in Charts II-2a and II-2b. These punchouts are considered in SSMA studs with flange widths of 1.625 in. or less. Standard punchout sizes are 1.5 in by 4.5 in. for sections with depths of 3.5 inches or more and 0.75 in. by 4.5 in. for sections with shallower depths. Punchouts are not included in the calculations for sections with web h/t ratios in excess of 200 due to the limits in Section B2.4. Those sections are marked with a "*".

(e) The flexural strengths were computed using the nominal yield stress listed in the charts, except for the SSMA sections (Charts II-2a and II-2b), the provisions of Section A7 of the *Specification* for strength increase from cold work of forming have been used where the section was eligible. Sections were considered eligible for the cold work of forming increase in yield stress if $\rho=1.0$ for each flat element, except that webs may have $\rho<1.0$ if the sum of b_1 plus b_2 from Section B2.3 equals or exceeds the width of the compression portion of the web.

(f) To obtain ASD design values, the nominal strengths in these charts must be divided by $\Omega=1.67$ (ASD).

(g) To obtain LRFD design values, the nominal strengths at the extreme left side of each curve, in the horizontal region where the section is not subject to lateral-torsional buckling, must be multiplied by $\phi=0.95$. To the right of the horizontal region of each curve, in the sloped region where the section is subject to lateral-torsional buckling, the nominal strengths must be multiplied by $\phi=0.90$.

(h) A solid line indicates that the section is the lightest available in the chart for a given nominal strength and unbraced length. A dashed line indicates that there is a lighter section available in the chart for that combination of nominal strength and unbraced length.

(i) A diamond on a curve designates the smallest possible distortional buckling strength and the corresponding lateral-torsional buckling length for that section calculated using Section C3.1.4(b). A more refined analysis using Sections C3.1.4(b) or C3.1.4(c) may give a larger available distortional buckling strength in cases where significant distortional bracing is present, or a significant moment gradient exists, or the spacing of discrete distortional braces is smaller than L_{cr}.

(j) Unbraced lengths in these charts are arbitrarily limited to a maximum of 40 times the depth of the section.

(k) Shear, web crippling, combined bending and shear, combined bending and web crippling and deflection must also be checked and they are not considered in these charts.

1.6 Beam Charts

Chart II-1a

Nominal Flexural Strength
C-Sections with Lips (F_y = 33 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)

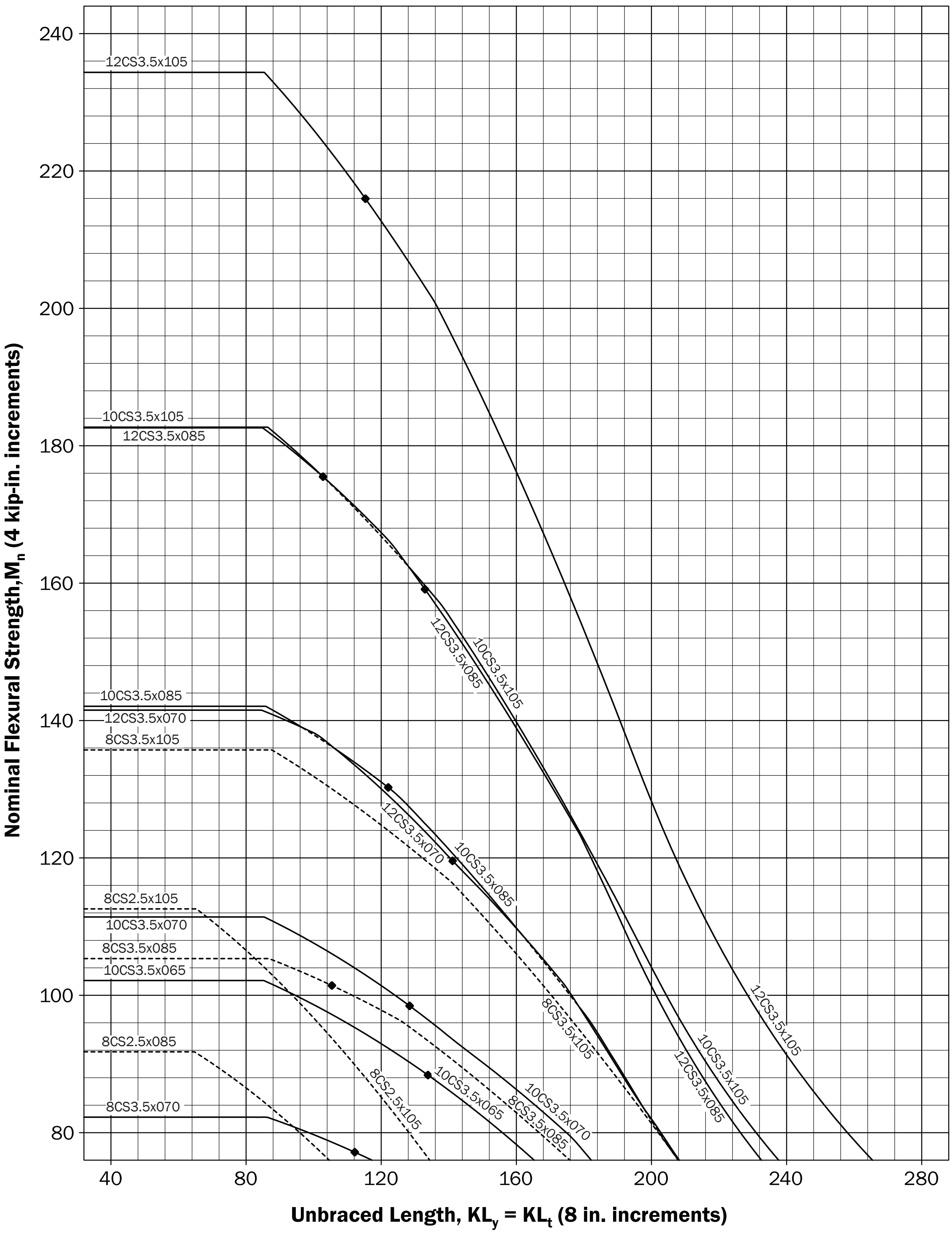
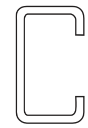


Chart II-1a

Nominal Flexural Strength
C-Sections with Lips (F_y = 33 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)



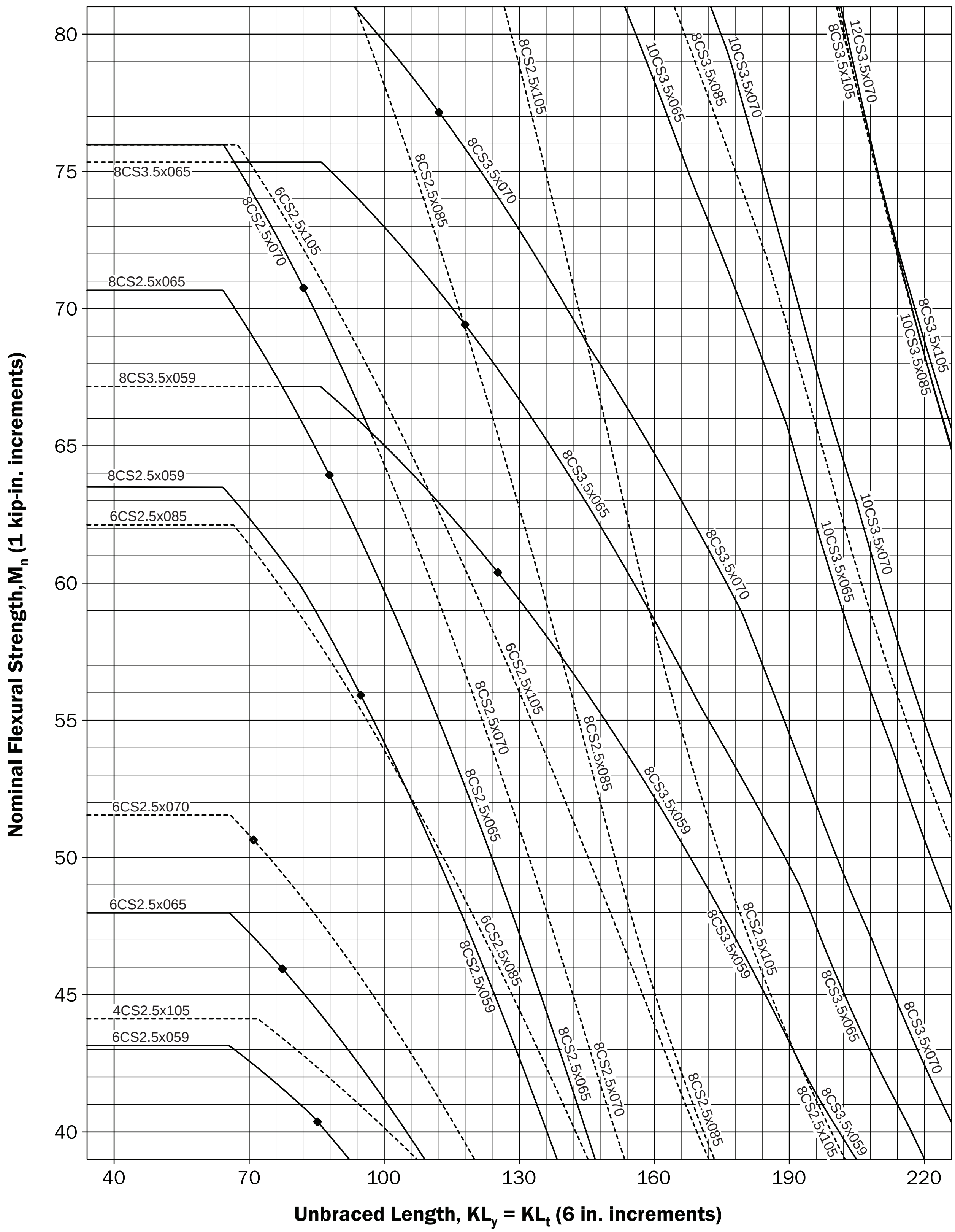


Chart II-1a

Nominal Flexural Strength
C-Sections with Lips (F_y = 33 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)

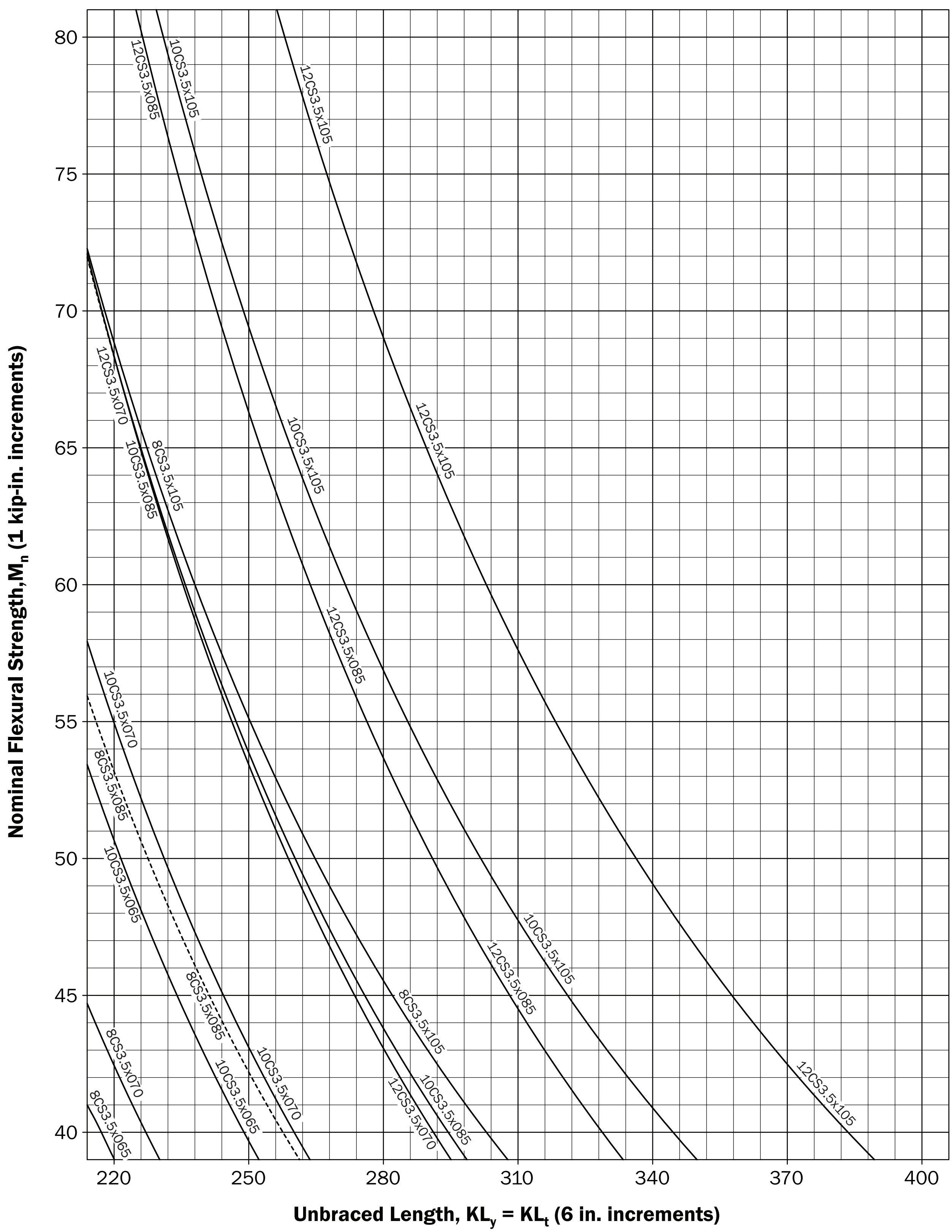


Chart II-1a

Nominal Flexural Strength
C-Sections with Lips (F_y = 33 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)

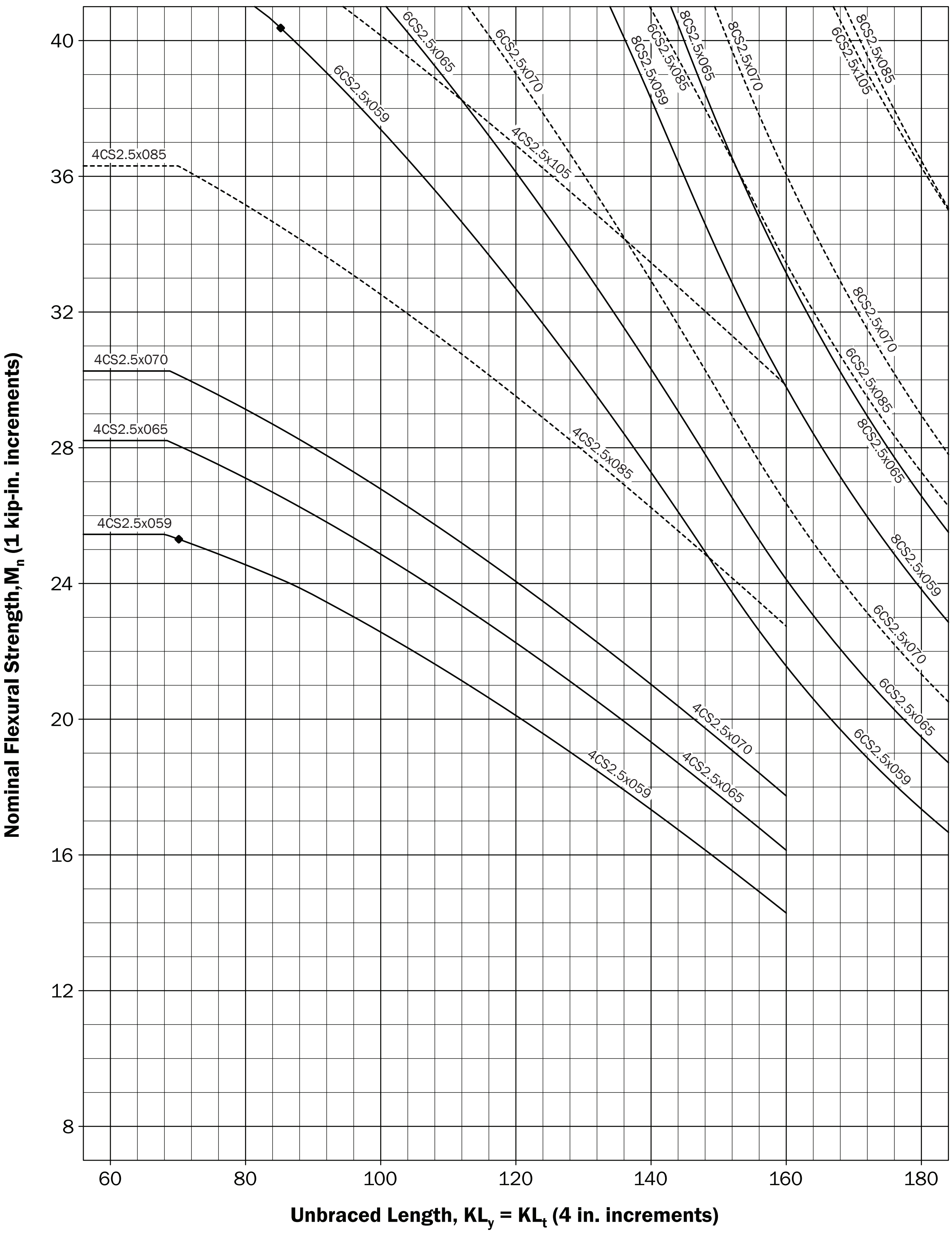
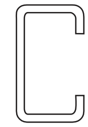


Chart II-1a

Nominal Flexural Strength
C-Sections with Lips (F_y = 33 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)



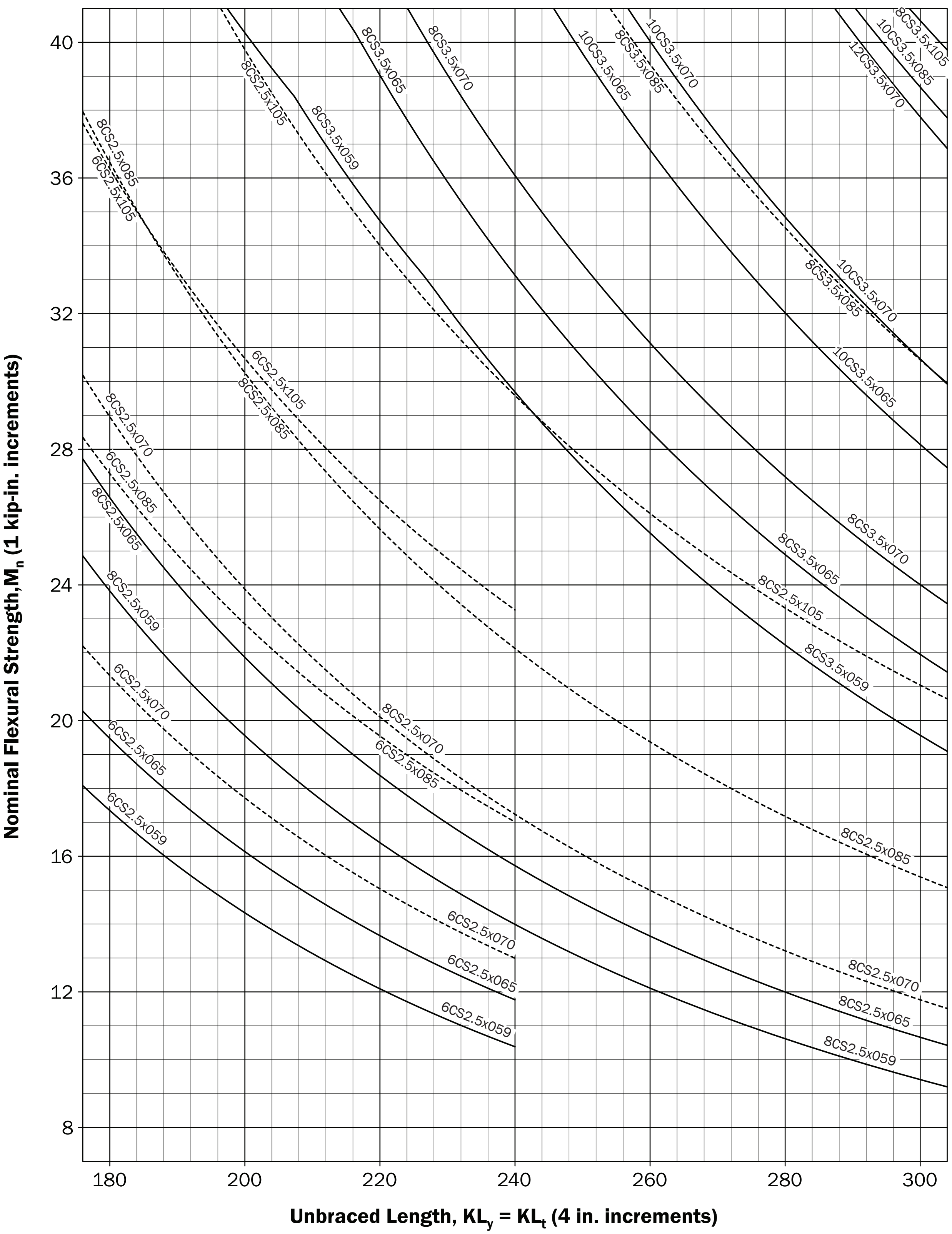


Chart II-1a

Nominal Flexural Strength
C-Sections with Lips (F_y = 33 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)

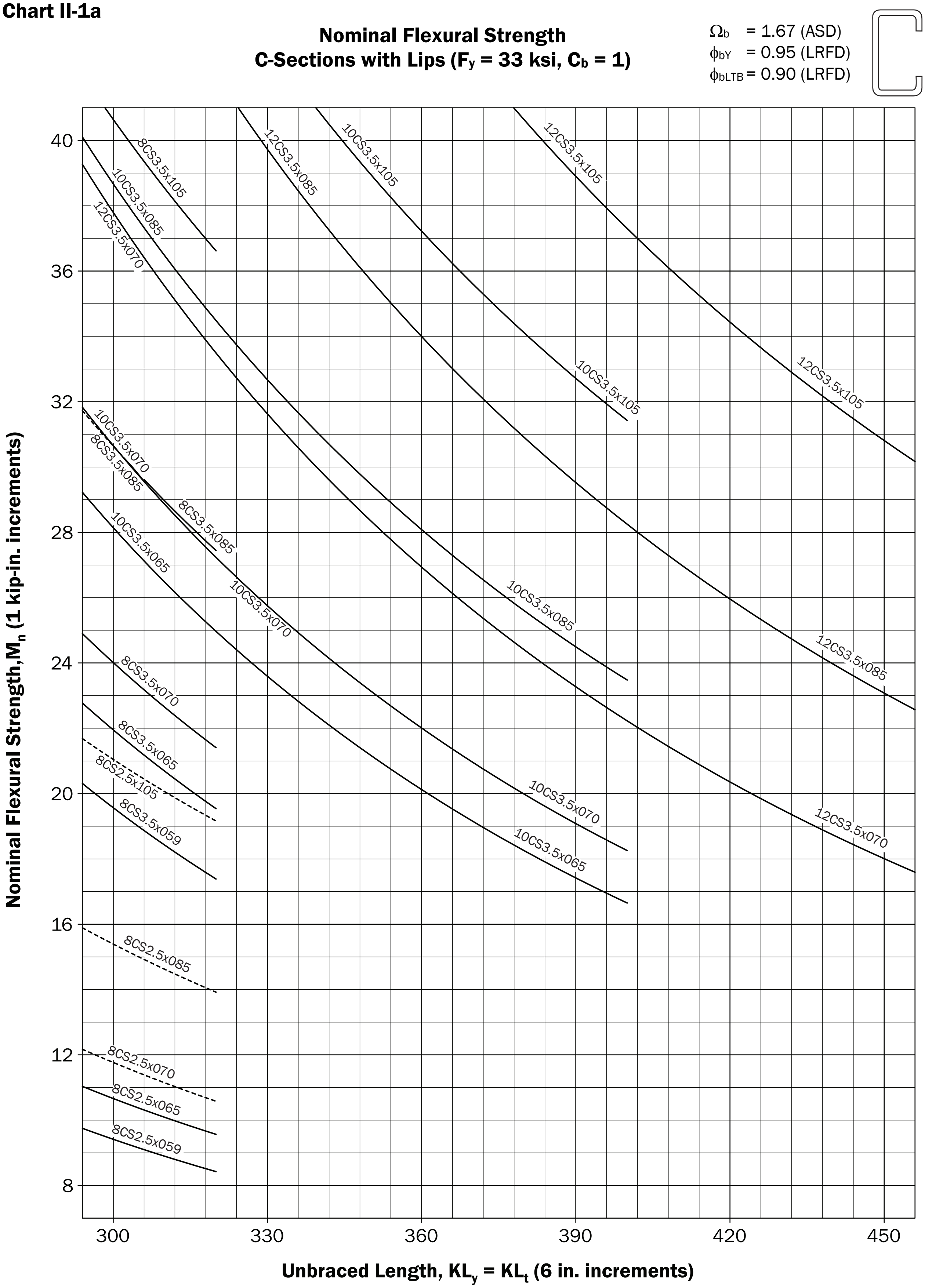


Chart II-1b

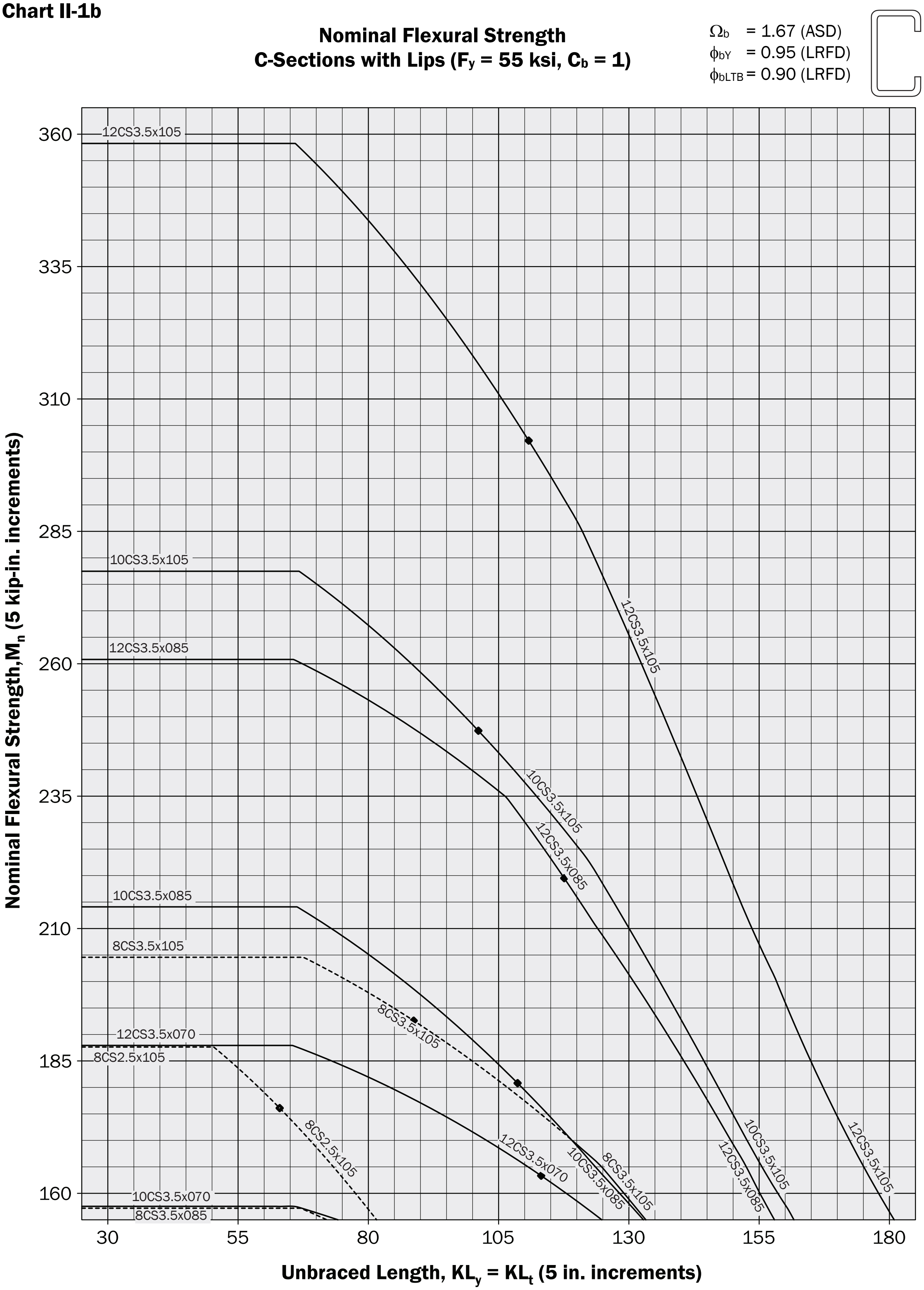

Nominal Flexural Strength
C-Sections with Lips (F_y = 55 ksi, C_b = 1)
Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)
Nominal Flexural Strength, M_n (5 kip-in. increments)
Unbraced Length, $KL_y = KL_t$ (5 in. increments)
360
335
310
285
260
235
210
185
160
30
55
80
105
130
155
180
12CS3.5x105
10CS3.5x105
12CS3.5x085
10CS3.5x085
8CS3.5x105
12CS3.5x070
8CS2.5x105
10CS3.5x070
8CS3.5x085

Chart II-1b

Nominal Flexural Strength
C-Sections with Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)

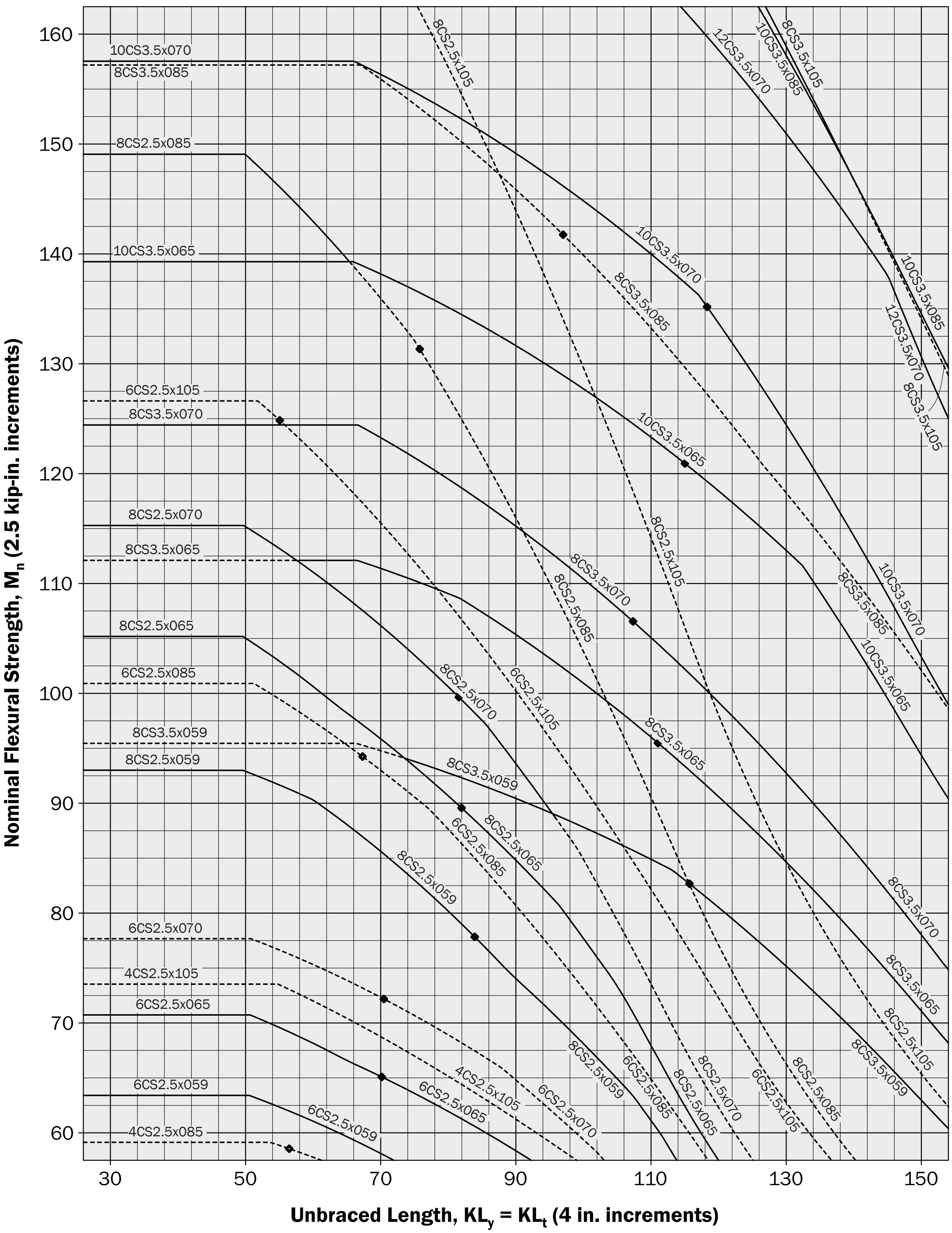


Chart II-1b

Nominal Flexural Strength
C-Sections with Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)

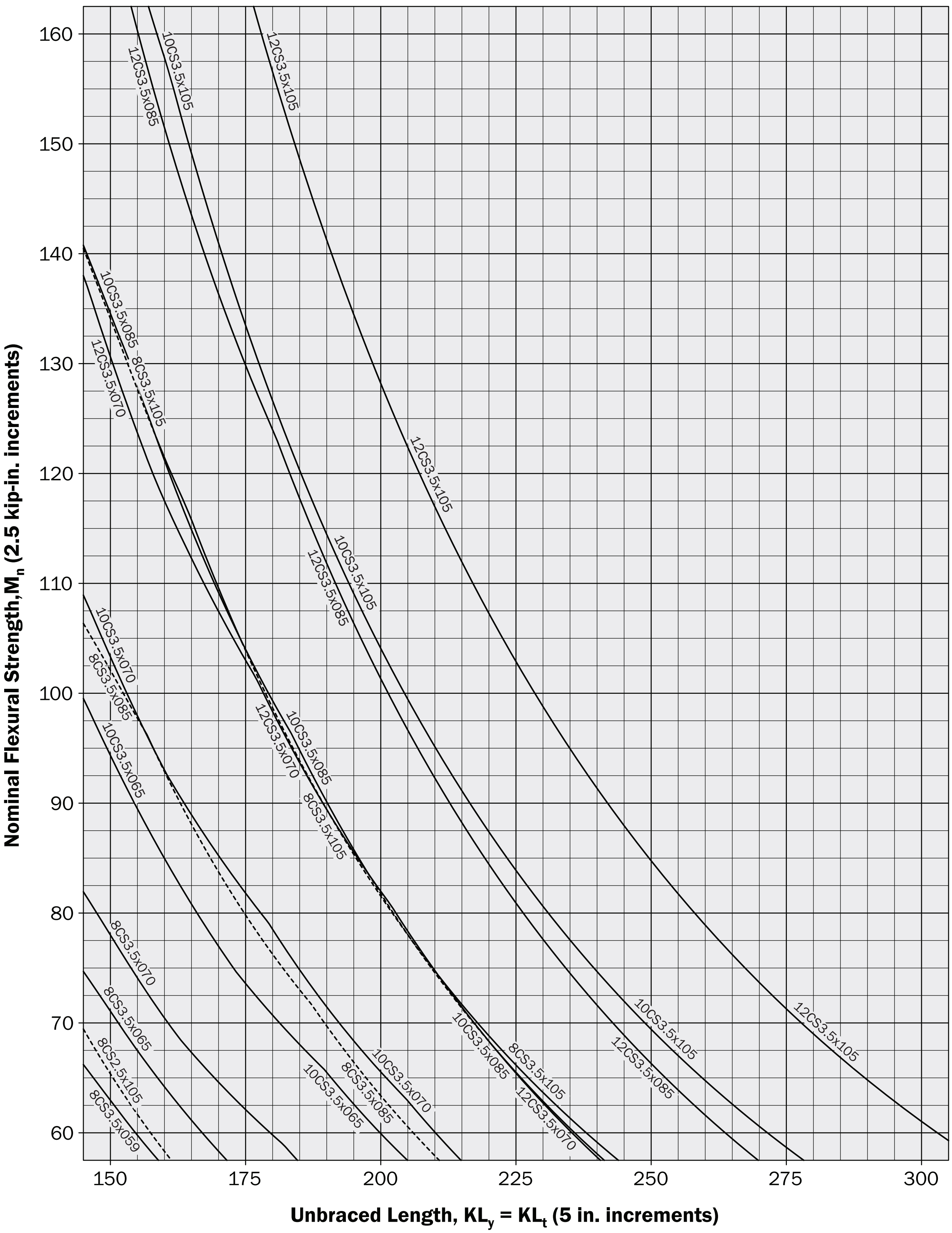


Chart II-1b

Nominal Flexural Strength
C-Sections with Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)

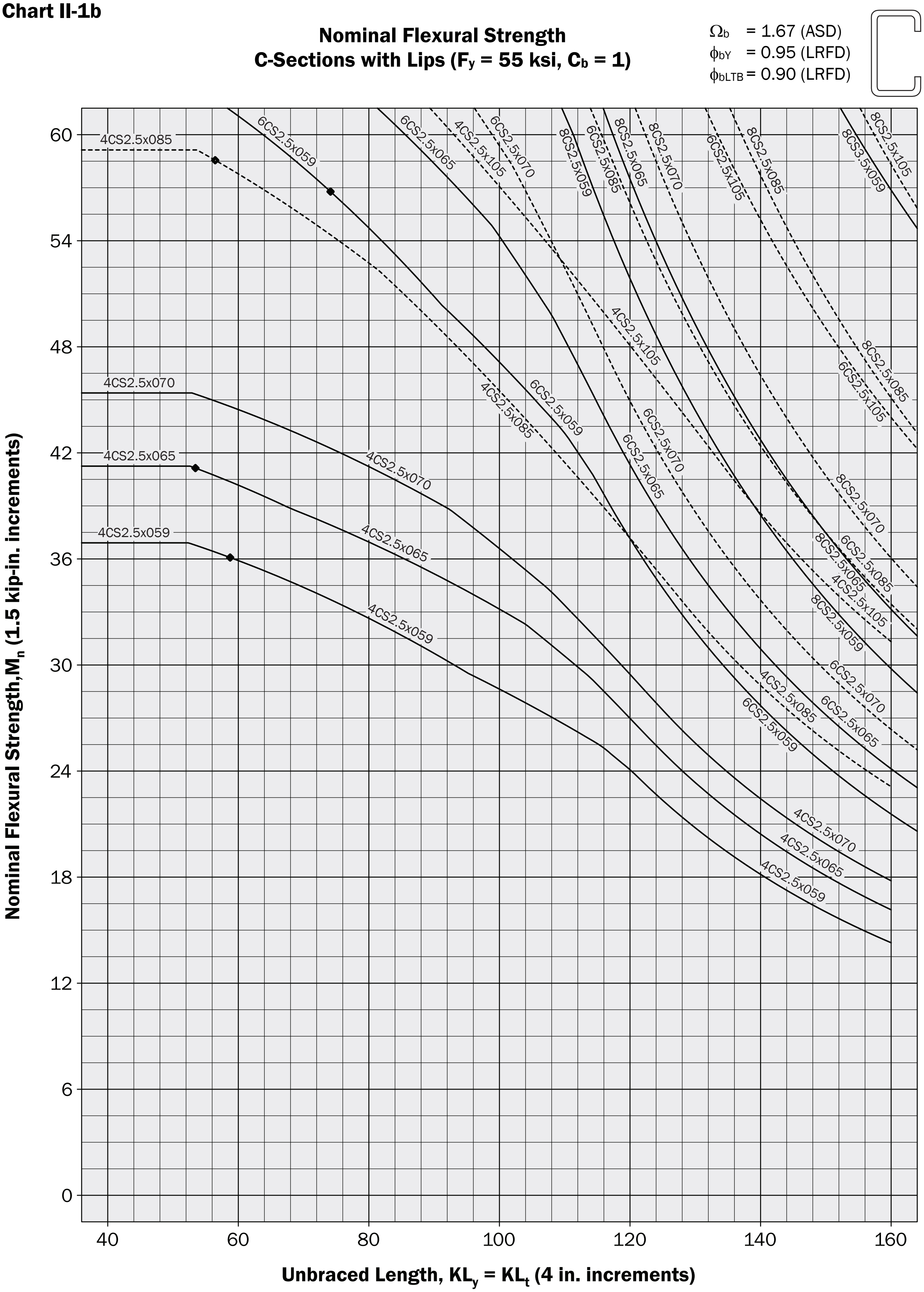


Chart II-1b

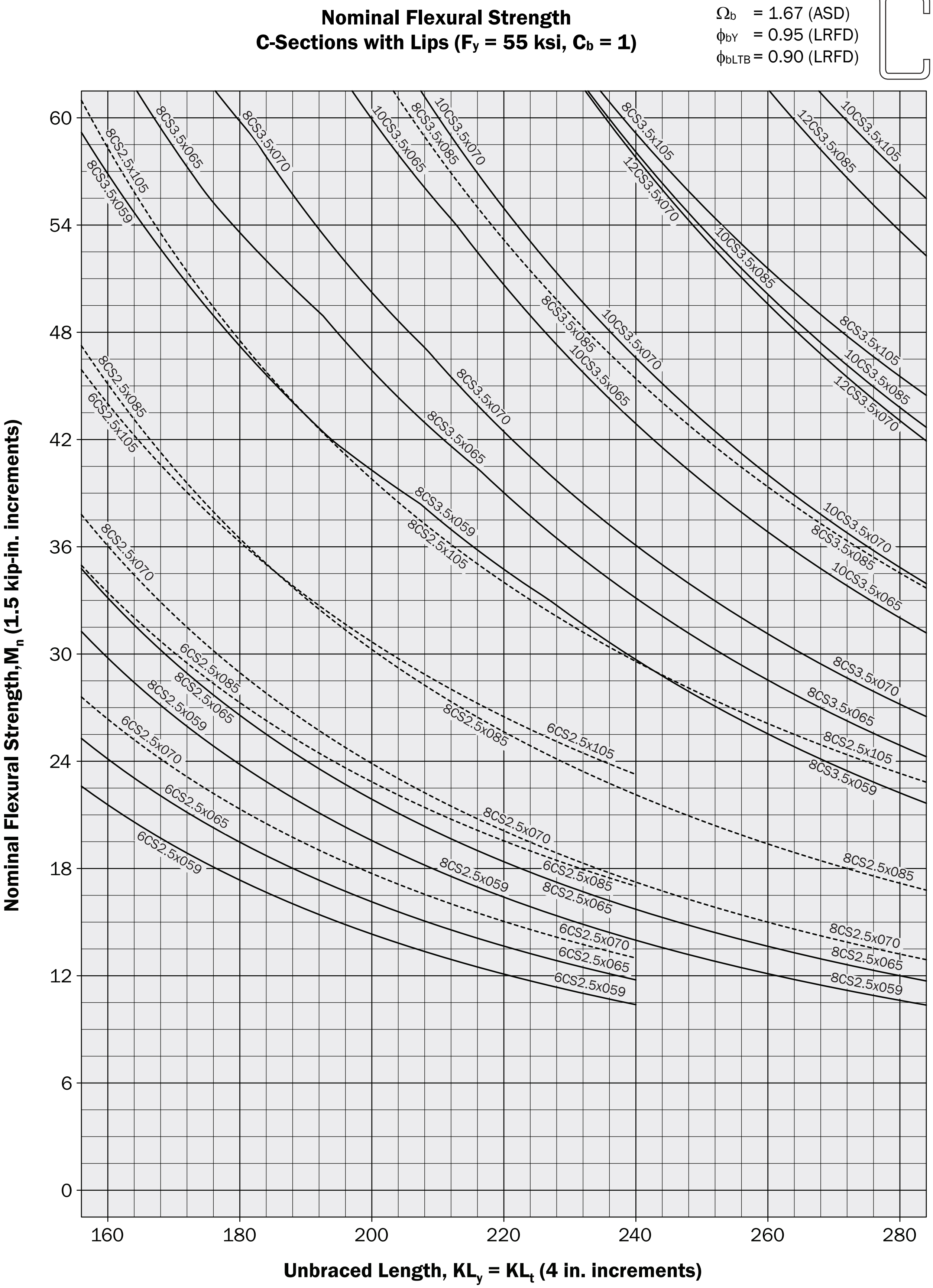

Nominal Flexural Strength
C-Sections with Lips (F_y = 55 ksi, C_b = 1)
Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)
Nominal Flexural Strength, M_n (1.5 kip-in. increments)
Unbraced Length, $KL_y = KL_t$ (4 in. increments)
0
6
12
18
24
30
36
42
48
54
60
160
180
200
220
240
260
280
8CS2.5x105
8CS3.5x059
8CS3.5x065
8CS3.5x070
10CS3.5x065
8CS3.5x085
10CS3.5x070
8CS3.5x105
12CS3.5x070
12CS3.5x085
10CS3.5x105
10CS3.5x085
8CS2.5x085
6CS2.5x105
8CS2.5x070
6CS2.5x085
8CS2.5x065
8CS2.5x059
6CS2.5x070
6CS2.5x065
6CS2.5x059
8CS2.5x085
6CS2.5x105
8CS2.5x070
8CS2.5x059
6CS2.5x085
8CS2.5x065
6CS2.5x070
6CS2.5x065
6CS2.5x059
8CS3.5x059
8CS2.5x105
8CS3.5x065
8CS3.5x070
8CS3.5x085
10CS3.5x065
10CS3.5x070
8CS3.5x105
10CS3.5x085
12CS3.5x070
8CS3.5x070
8CS3.5x065
8CS2.5x105
8CS3.5x059
8CS2.5x085
8CS2.5x070
8CS2.5x065
8CS2.5x059

Chart II-1b

Nominal Flexural Strength
C-Sections with Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)

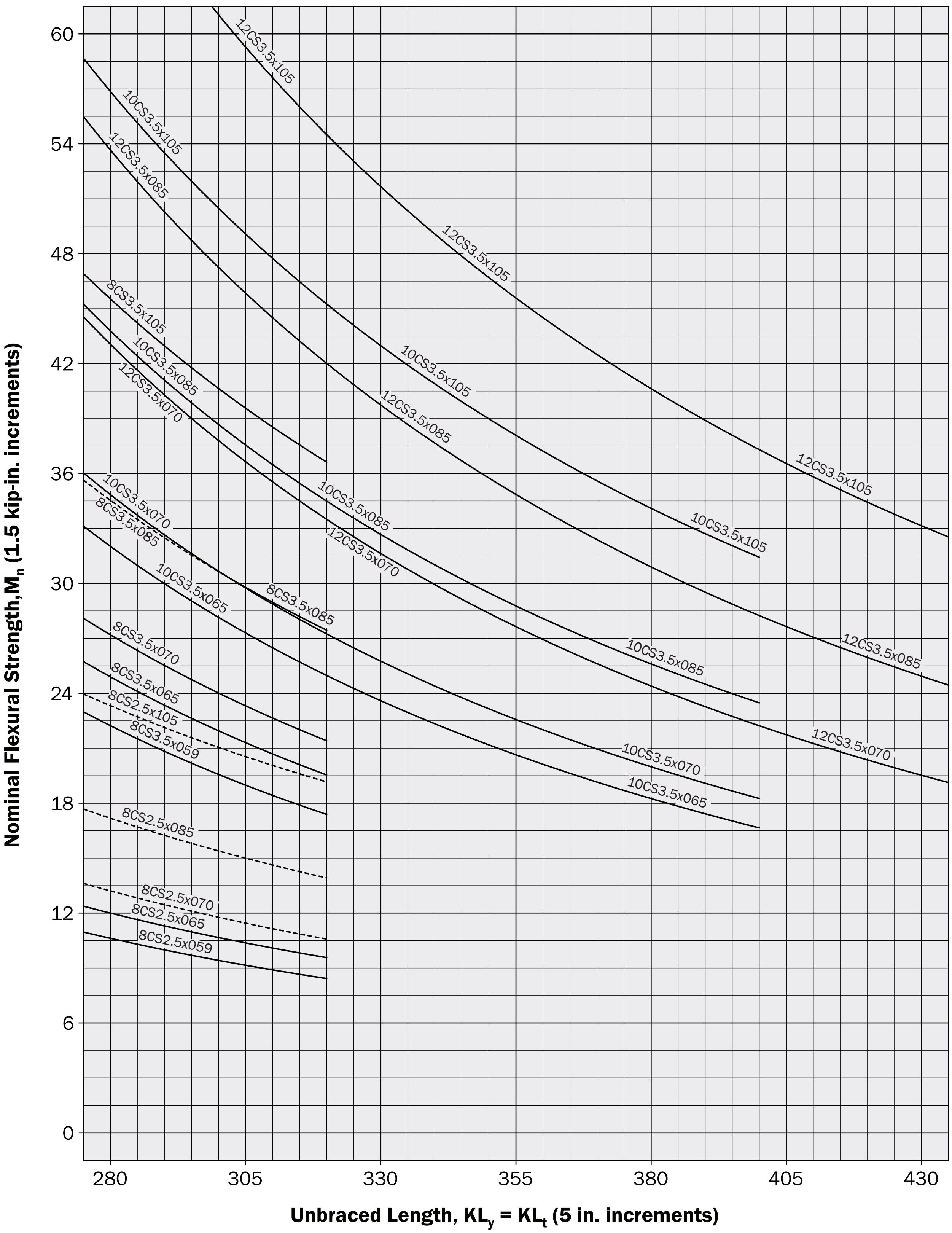
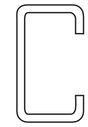


Chart II-2a

Nominal Flexural Strength
SSMA Studs
C-Sections with Lips (F_y = 33 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)



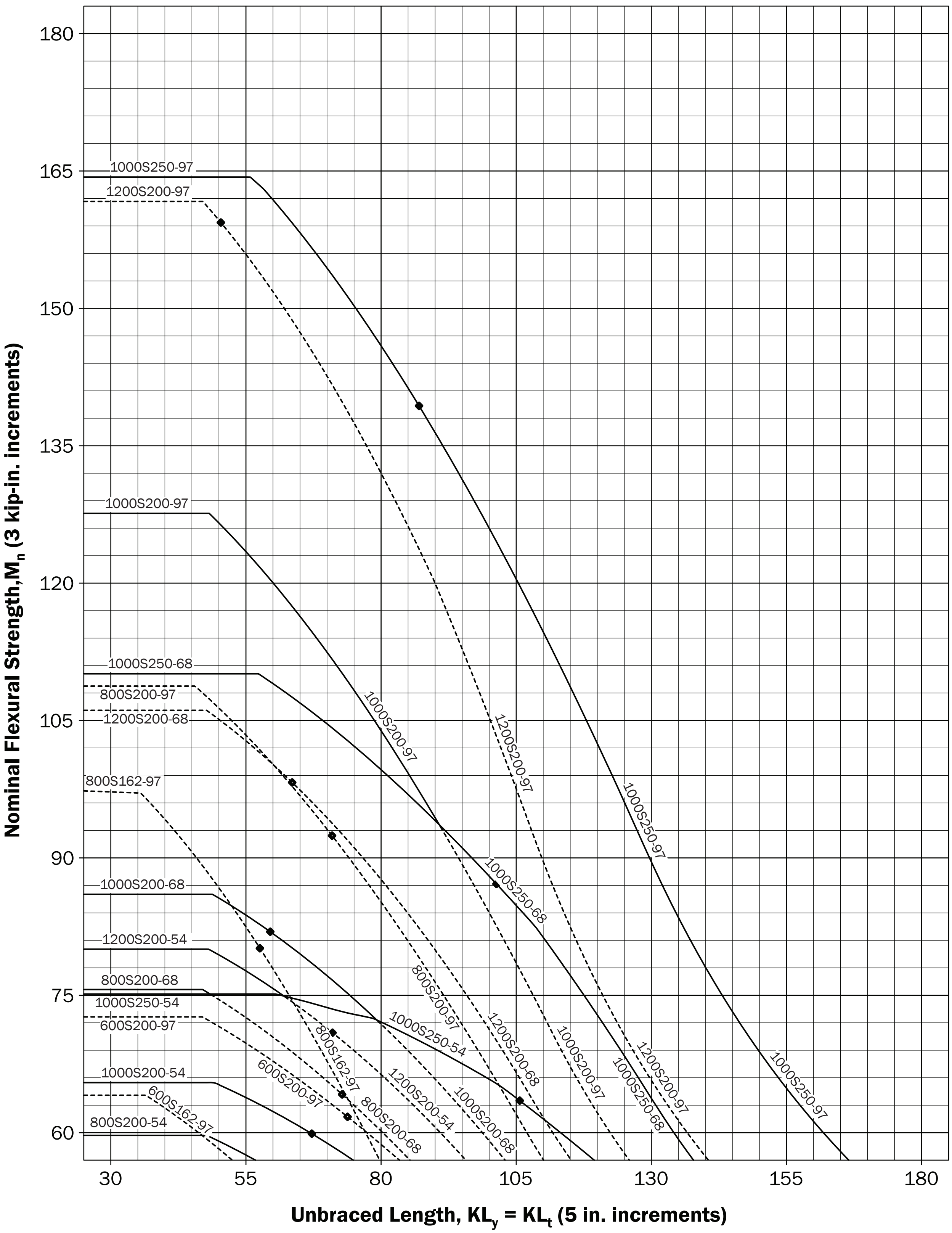


Chart II-2a

Nominal Flexural Strength
SSMA Studs
C-Sections with Lips (F_y = 33 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)

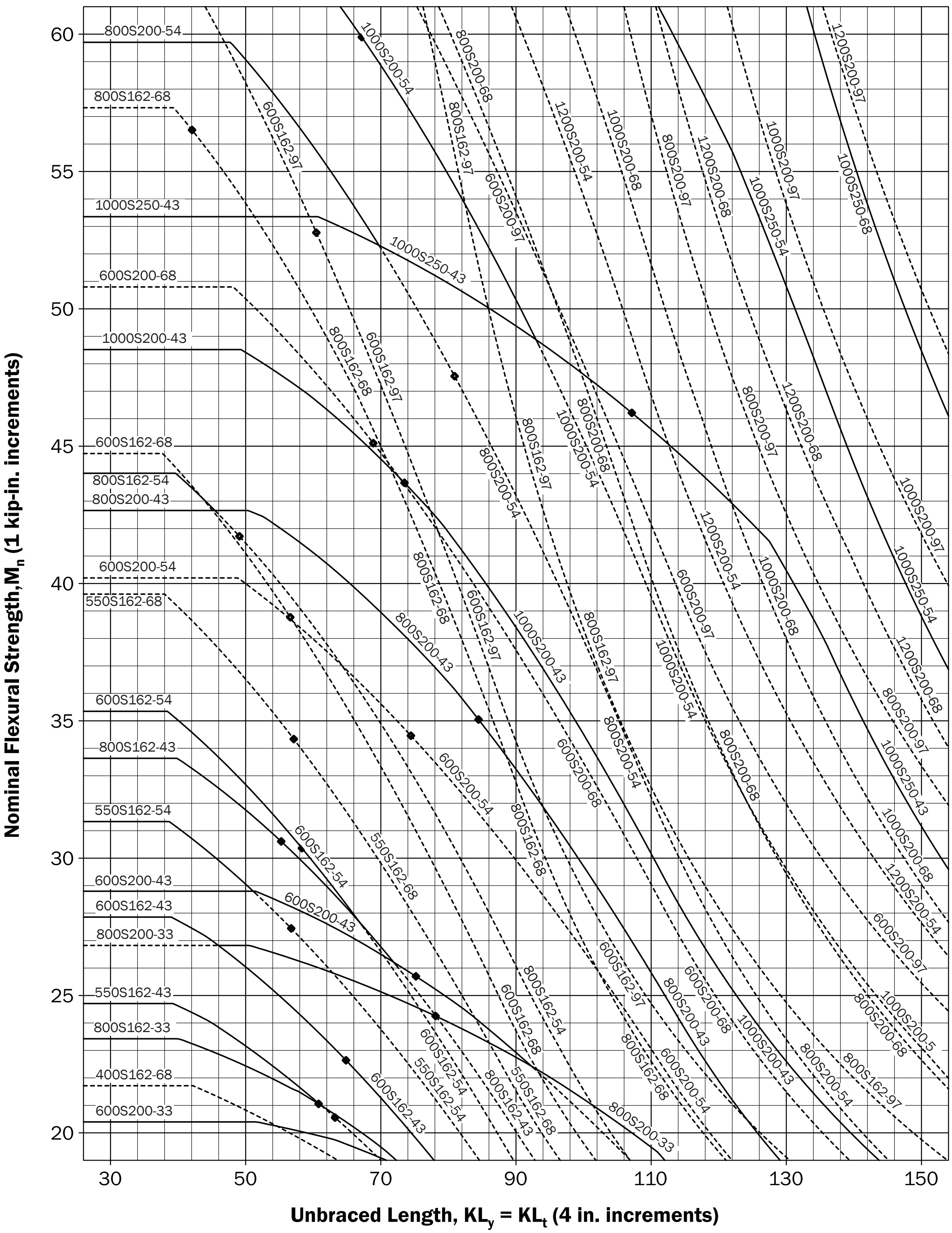


Chart II-2a

Nominal Flexural Strength
SSMA Studs
C-Sections with Lips (F_y = 33 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)

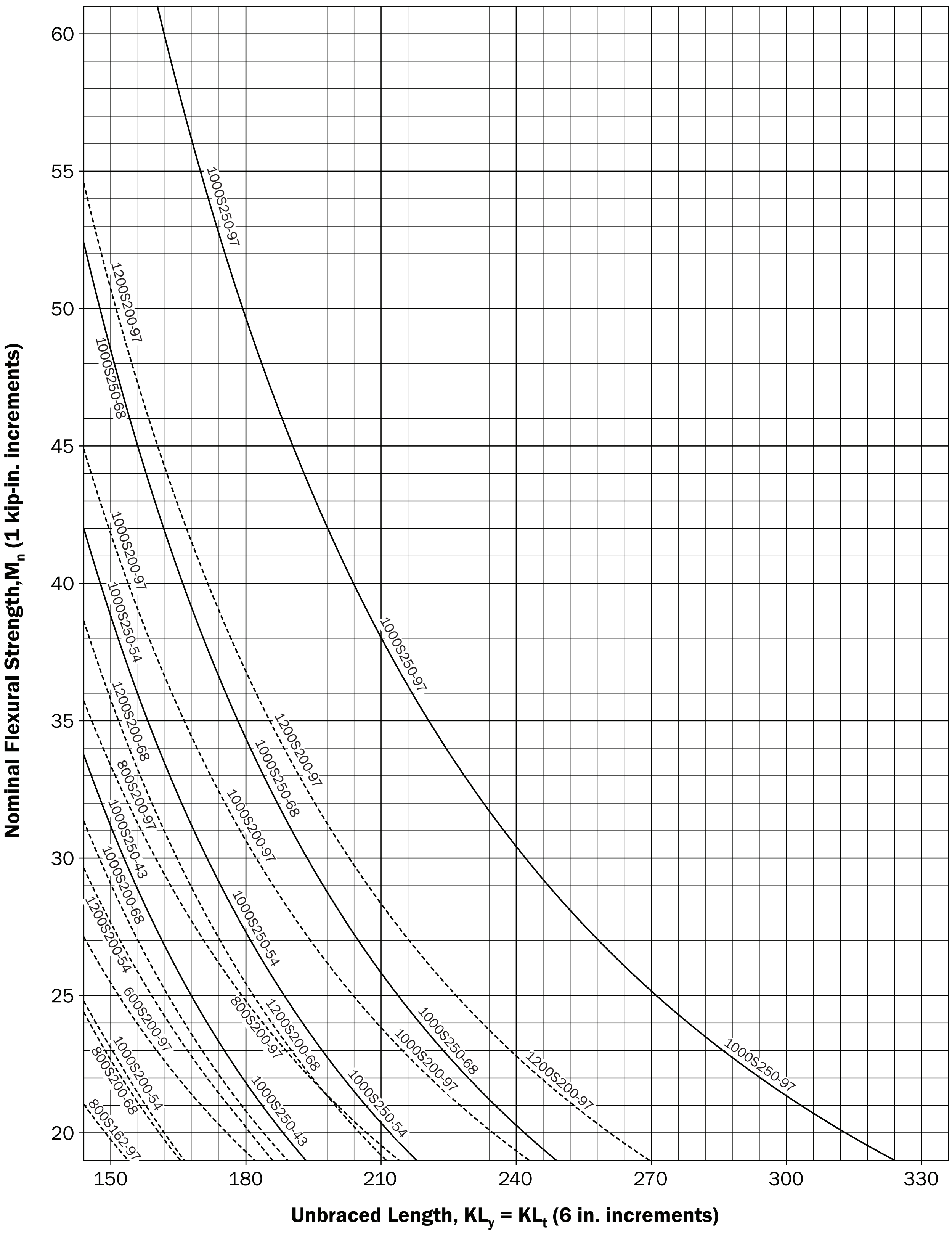
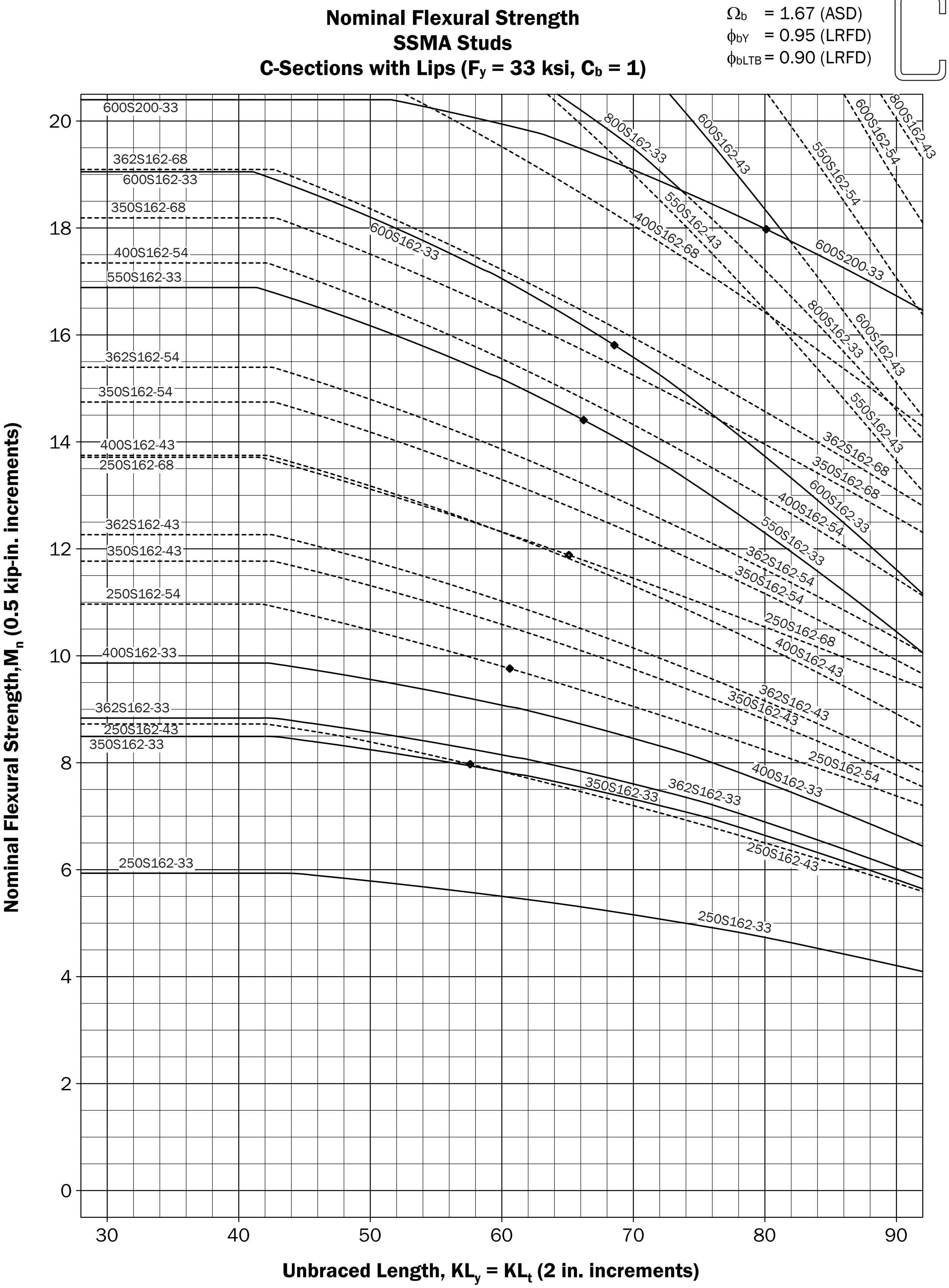


Chart II-2a



Nominal Flexural Strength
SSMA Studs
C-Sections with Lips (F_y = 33 ksi, C_b = 1)
Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)
600S200-33
362S162-68
600S162-33
350S162-68
400S162-54
550S162-33
362S162-54
350S162-54
400S162-43
250S162-68
362S162-43
350S162-43
250S162-54
400S162-33
362S162-33
250S162-43
350S162-33
250S162-33
800S162-33
600S162-43
550S162-54
600S162-54
800S162-43
550S162-43
400S162-68
Nominal Flexural Strength, M_n (0.5 kip-in. increments)
Unbraced Length, $KL_y = KL_t$ (2 in. increments)

Chart II-2a

Nominal Flexural Strength
SSMA Studs
C-Sections with Lips (F_y = 33 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)

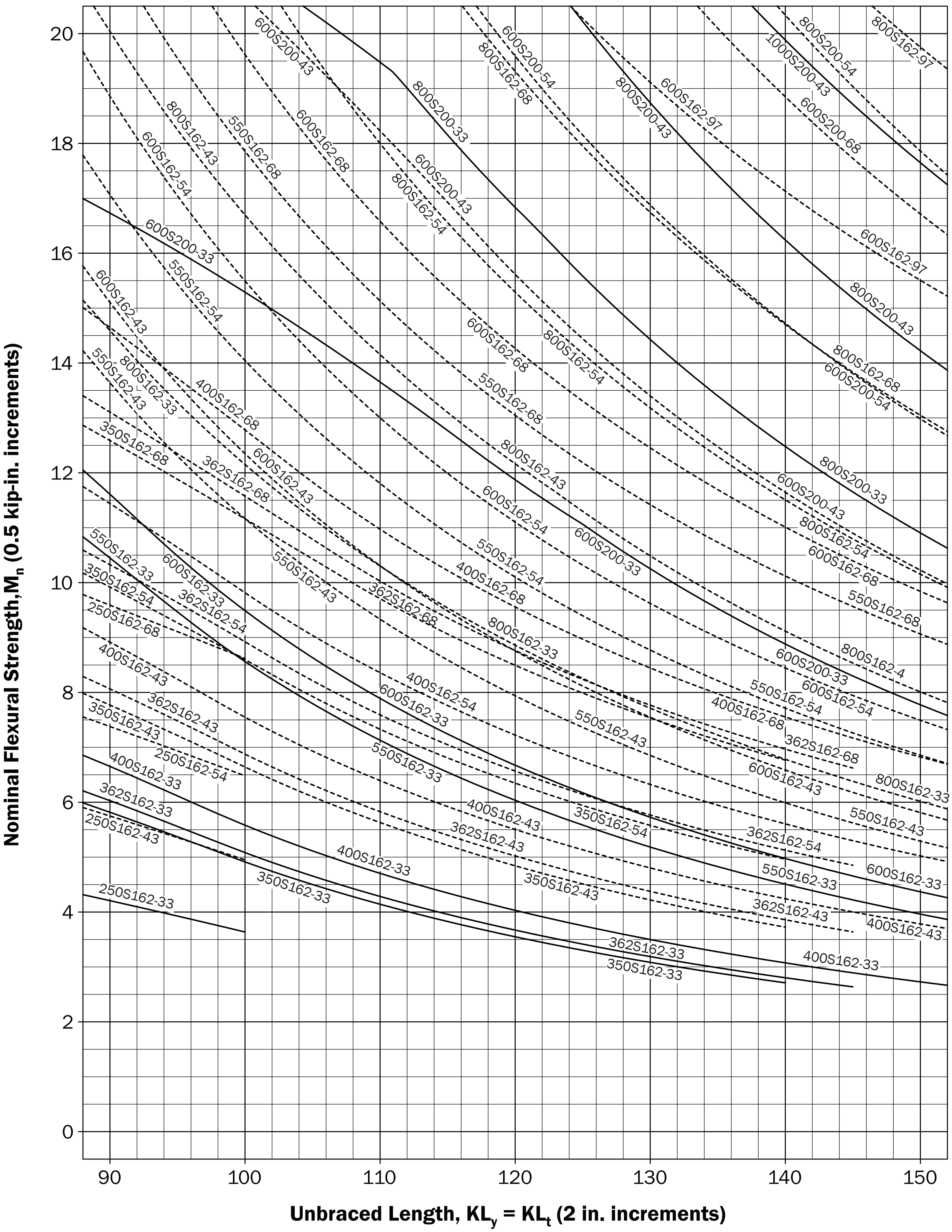


Chart II-2a

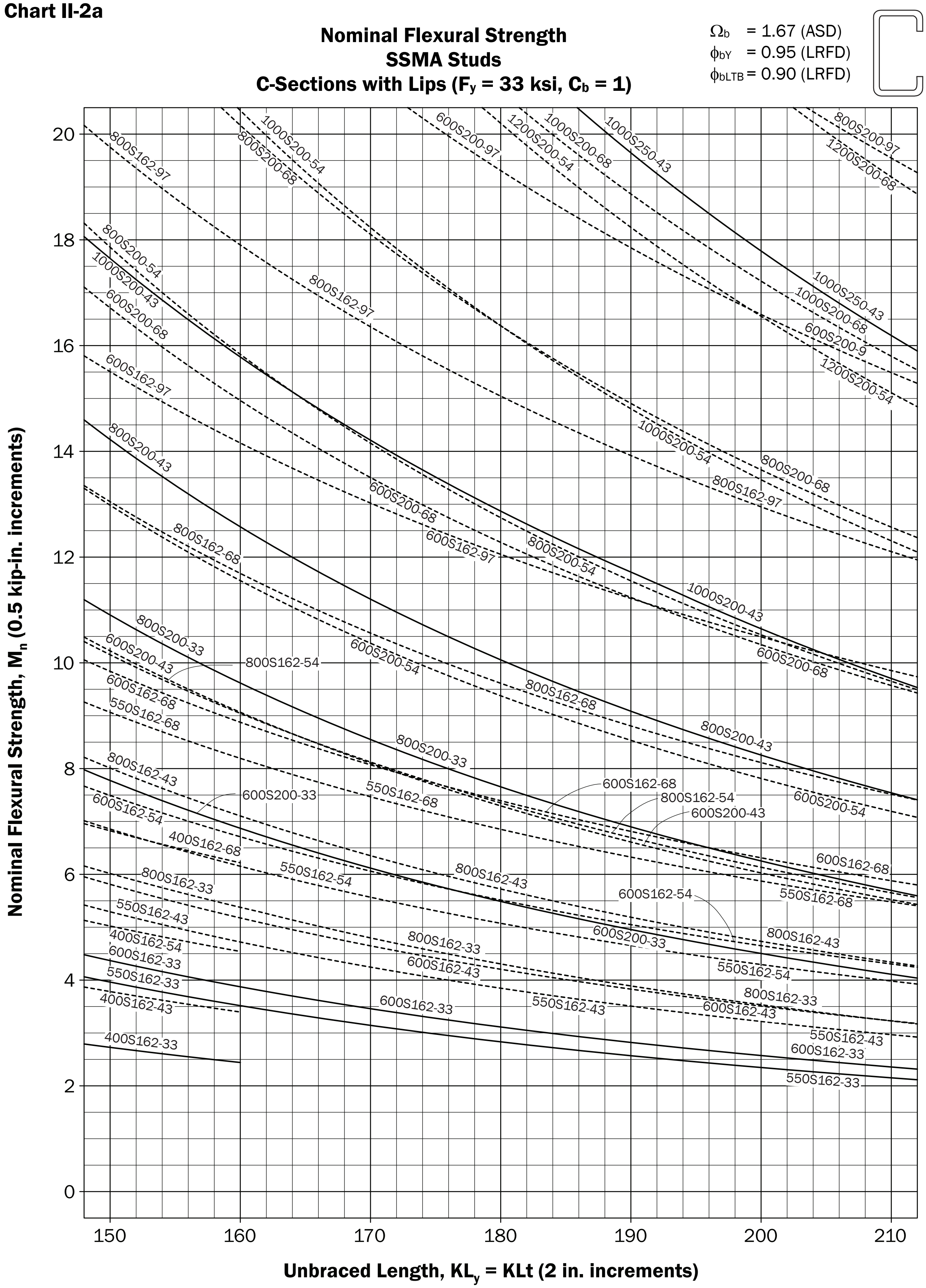

Nominal Flexural Strength
SSMA Studs
C-Sections with Lips (Fy = 33 ksi, Cb = 1)
Ωb = 1.67 (ASD)
φbY = 0.95 (LRFD)
φbLTB = 0.90 (LRFD)
Nominal Flexural Strength, Mn (0.5 kip-in. increments)
Unbraced Length, KLy = KLt (2 in. increments)
0
2
4
6
8
10
12
14
16
18
20
150
160
170
180
190
200
210
800S162-97
1000S200-54
800S200-68
600S200-97
1200S200-54
1000S200-68
1000S250-43
800S200-97
1200S200-68
800S200-54
1000S200-43
600S200-68
800S162-97
1000S250-43
1000S200-68
600S200-9
1200S200-54
600S162-97
800S200-43
1000S200-54
800S200-68
800S162-97
600S200-68
600S162-97
800S200-54
800S162-68
1000S200-43
800S200-33
600S200-43
800S162-54
600S200-54
600S200-68
600S162-68
550S162-68
800S162-68
800S200-43
800S200-33
800S162-43
600S200-33
550S162-68
600S162-68
800S162-54
600S200-43
600S200-54
600S162-54
400S162-68
550S162-54
800S162-43
600S162-68
800S162-33
600S162-54
550S162-68
550S162-43
400S162-54
600S162-33
800S162-33
600S162-43
600S200-33
800S162-43
550S162-54
550S162-33
800S162-33
400S162-43
600S162-33
550S162-43
600S162-43
550S162-43
400S162-33
600S162-33
550S162-33

Chart II-2a

Nominal Flexural Strength
SSMA Studs
C-Sections with Lips (F_y = 33 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)

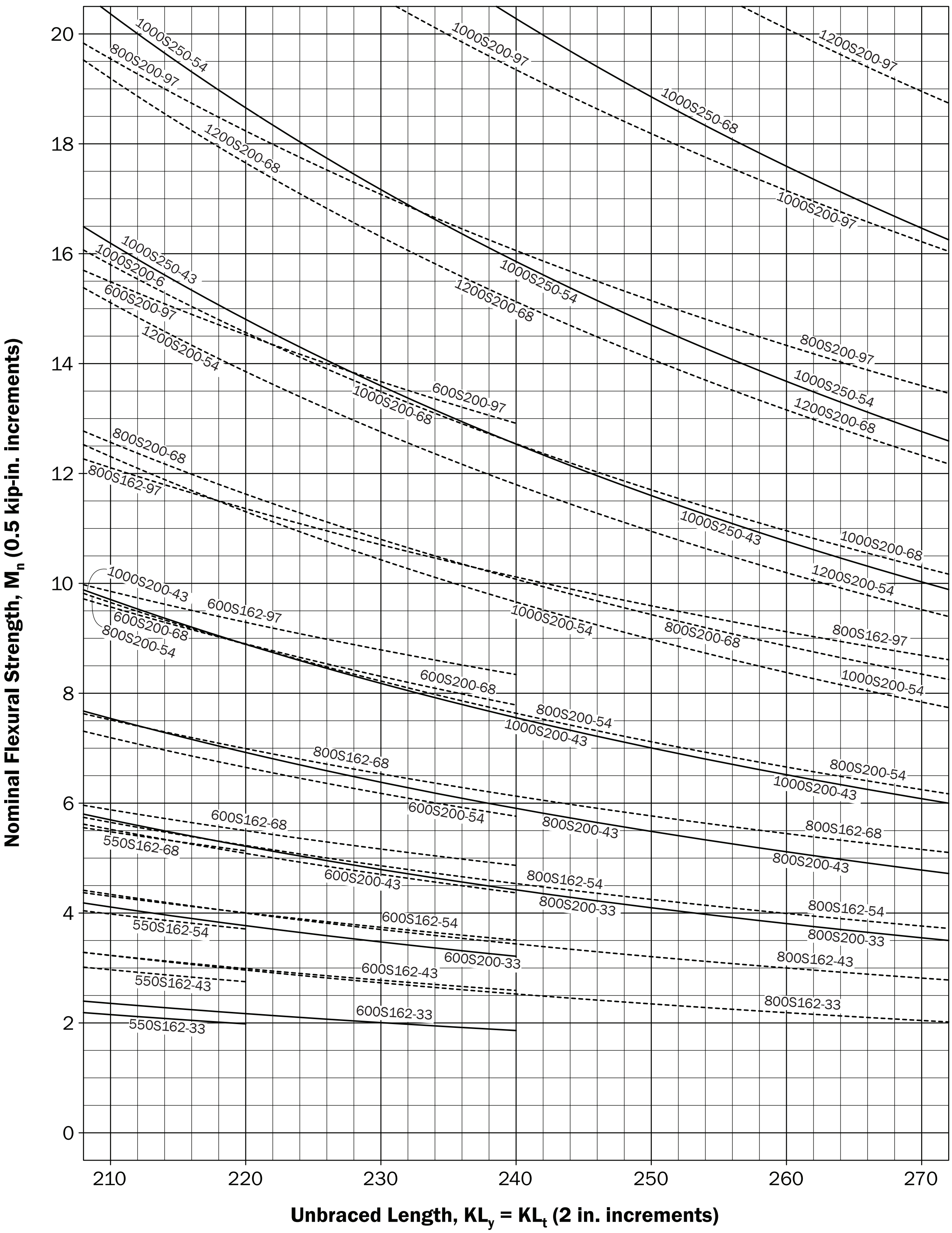


Chart II-2a

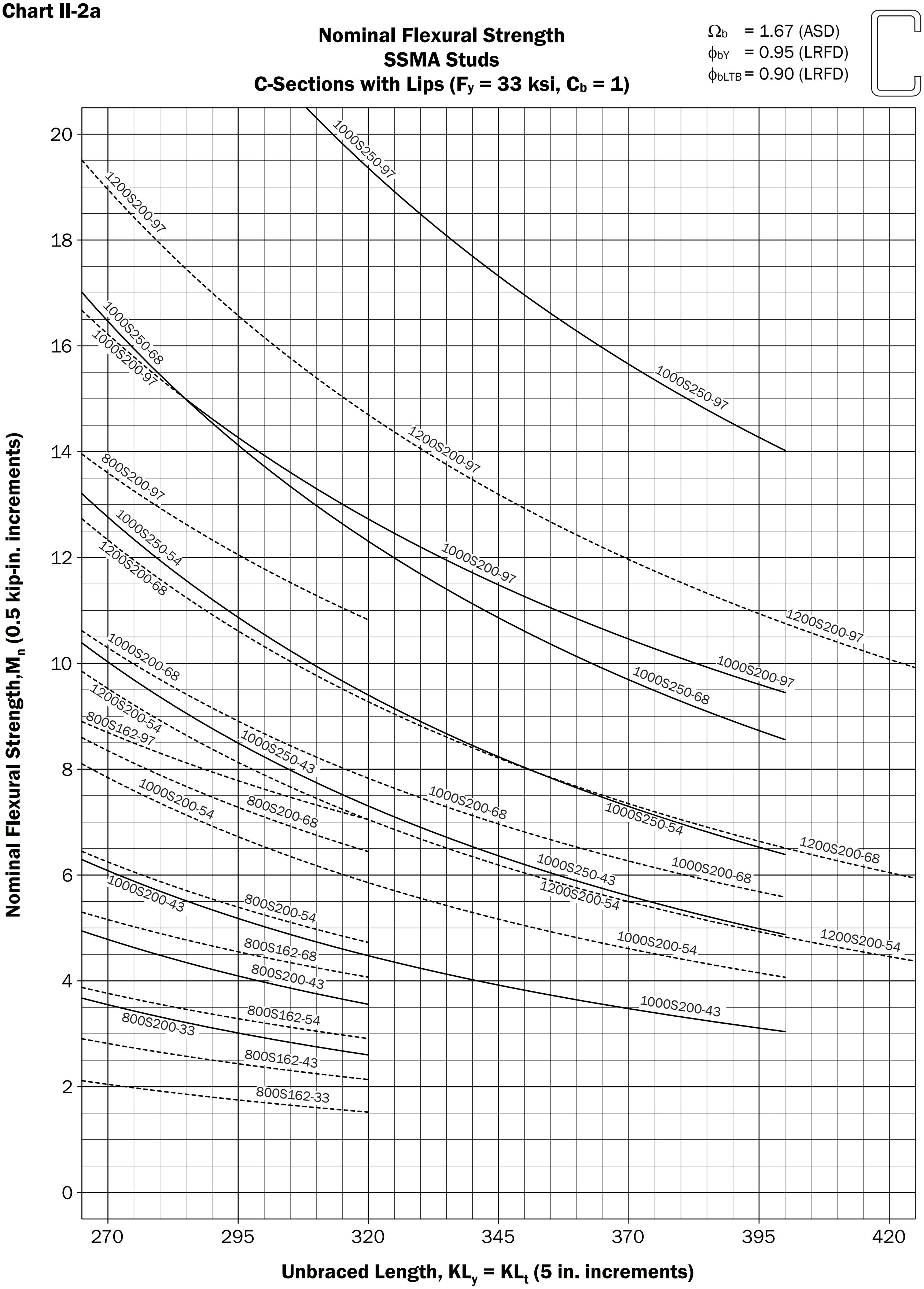

Nominal Flexural Strength
SSMA Studs
C-Sections with Lips (F_y = 33 ksi, C_b = 1)
Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)
Nominal Flexural Strength, M_n (0.5 kip-in. increments)
Unbraced Length, $KL_y = KL_t$ (5 in. increments)
0
2
4
6
8
10
12
14
16
18
20
270
295
320
345
370
395
420
1000S250-97
1200S200-97
1000S250-68
1000S200-97
800S200-97
1000S250-54
1200S200-68
1000S200-68
1200S200-54
800S162-97
1000S250-43
1000S200-54
800S200-68
1000S200-43
800S200-54
800S162-68
800S200-43
800S162-54
800S200-33
800S162-43
800S162-33

Chart II-2b

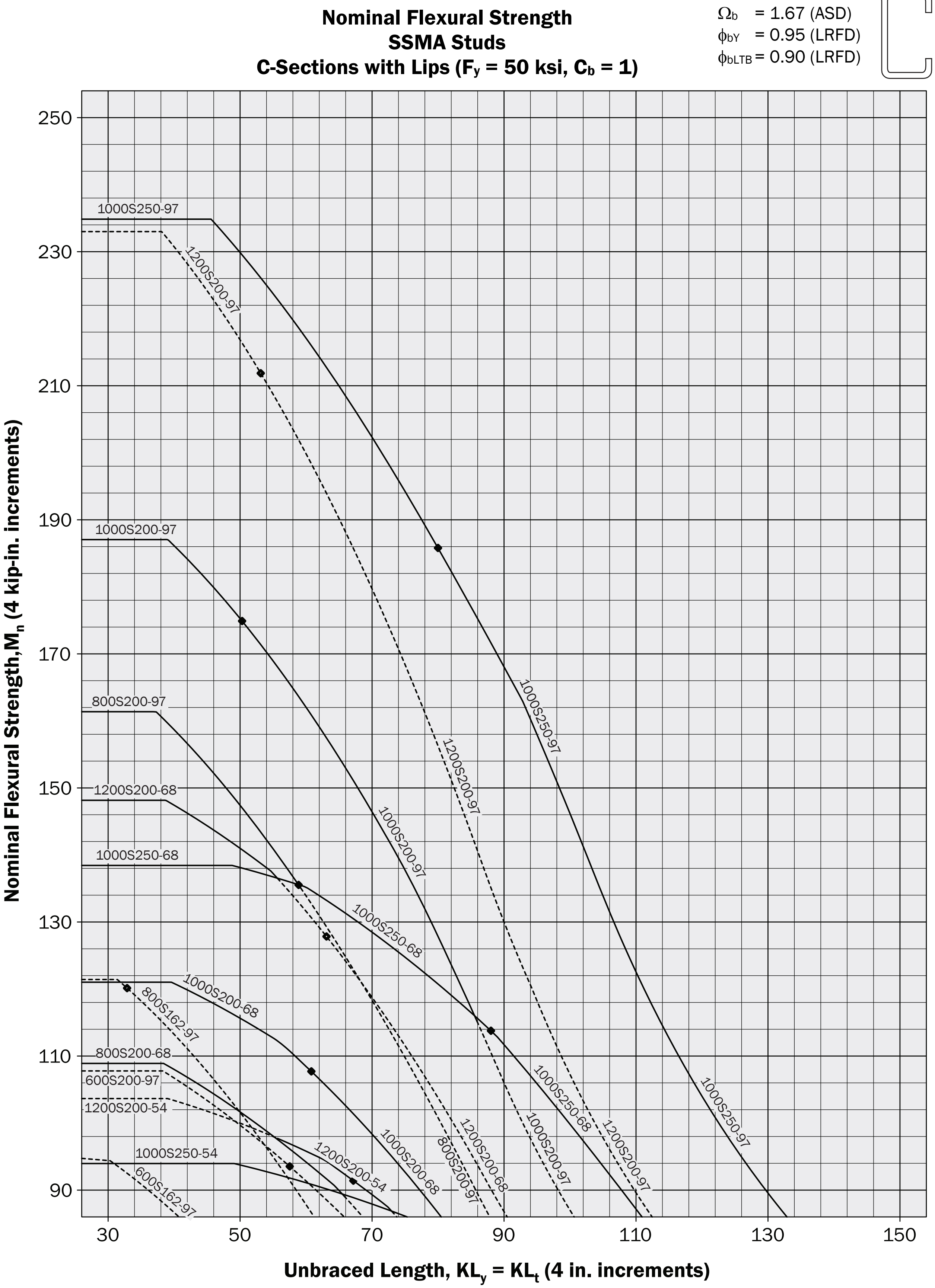

Nominal Flexural Strength
SSMA Studs
C-Sections with Lips (F_y = 50 ksi, C_b = 1)
Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)
Nominal Flexural Strength, M_n (4 kip-in. increments)
Unbraced Length, $KL_y = KL_t$ (4 in. increments)
250
230
210
190
170
150
130
110
90
30
50
70
90
110
130
150
1000S250-97
1200S200-97
1000S200-97
800S200-97
1200S200-68
1000S250-68
800S162-97
1000S200-68
800S200-68
600S200-97
1200S200-54
1000S250-54
600S162-97
1200S200-54
1000S200-68
800S200-97
1200S200-68
1000S200-97
1000S250-68
1200S200-97
1000S250-97

Chart II-2b

Nominal Flexural Strength
SSMA Studs
C-Sections with Lips (F_y = 50 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)

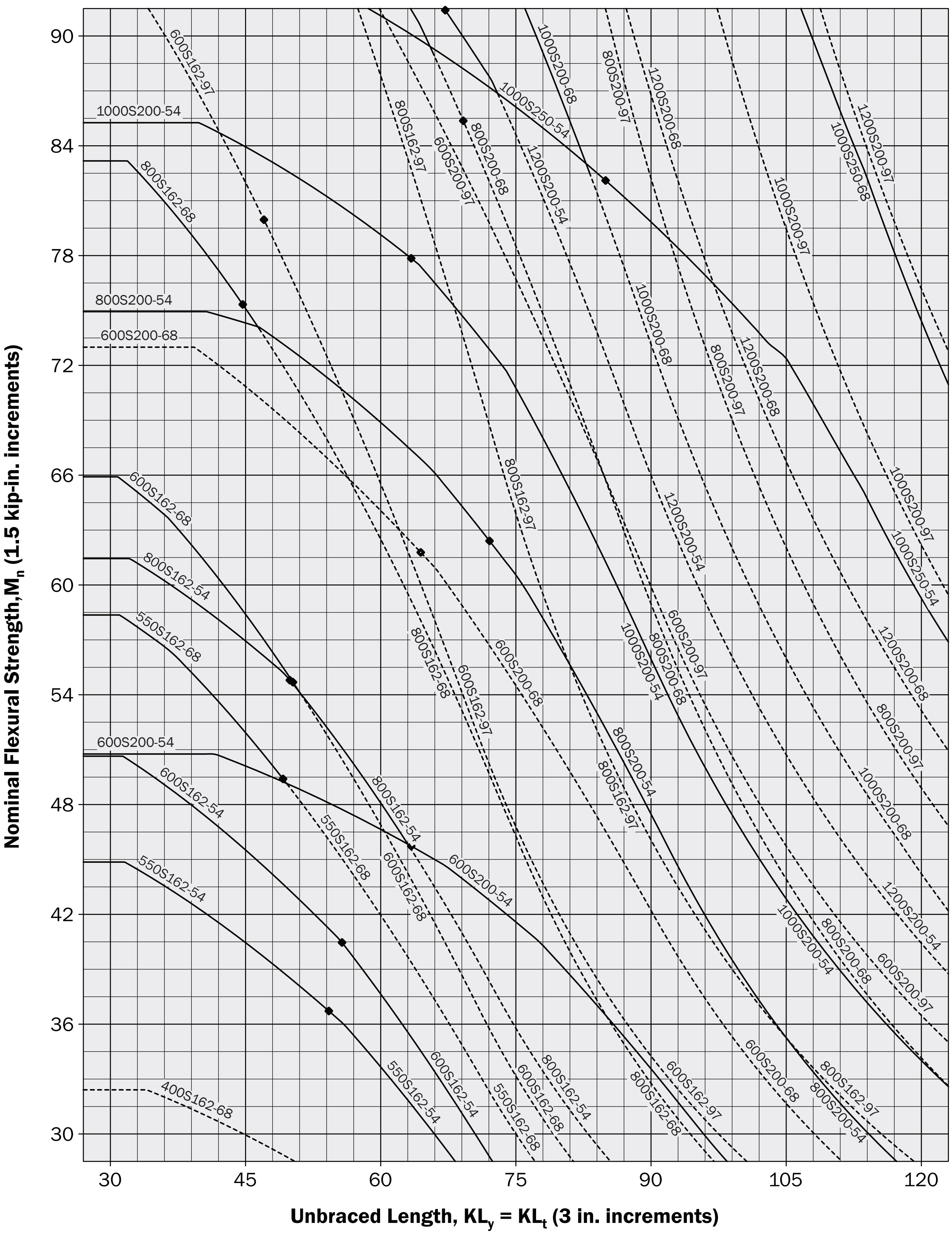


Chart II-2b

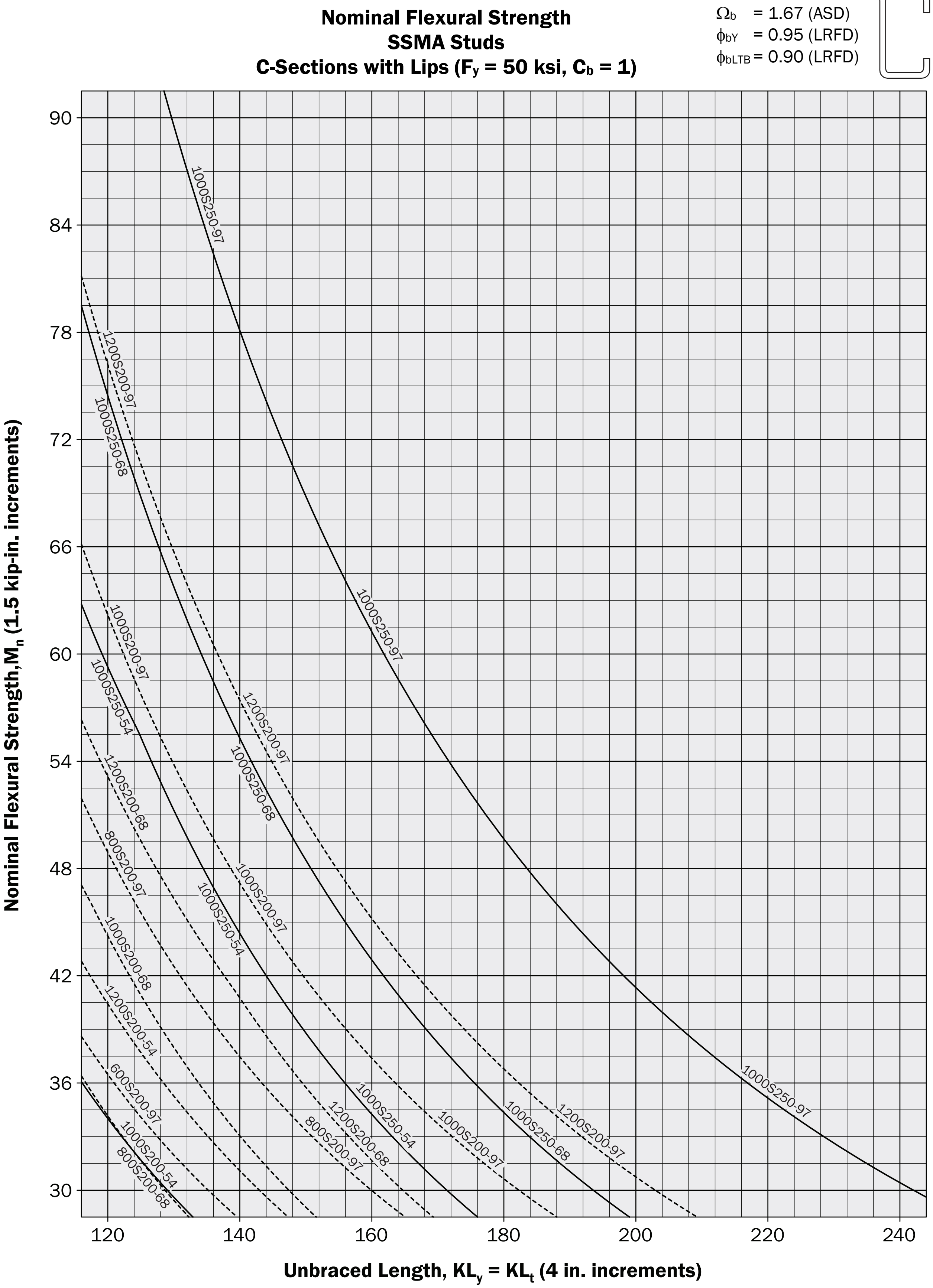

Nominal Flexural Strength
SSMA Studs
C-Sections with Lips (Fy = 50 ksi, Cb = 1)
Ωb = 1.67 (ASD)
φbY = 0.95 (LRFD)
φbLTB = 0.90 (LRFD)
Nominal Flexural Strength, Mn (1.5 kip-in. increments)
Unbraced Length, KLy = KLt (4 in. increments)
90
84
78
72
66
60
54
48
42
36
30
120
140
160
180
200
220
240
1000S250-97
1200S200-97
1000S250-68
1000S200-97
1000S250-54
1200S200-68
800S200-97
1000S200-68
1200S200-54
600S200-97
1000S200-54
800S200-68

Chart II-2b

Nominal Flexural Strength
SSMA Studs
C-Sections with Lips (F_y = 50 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)

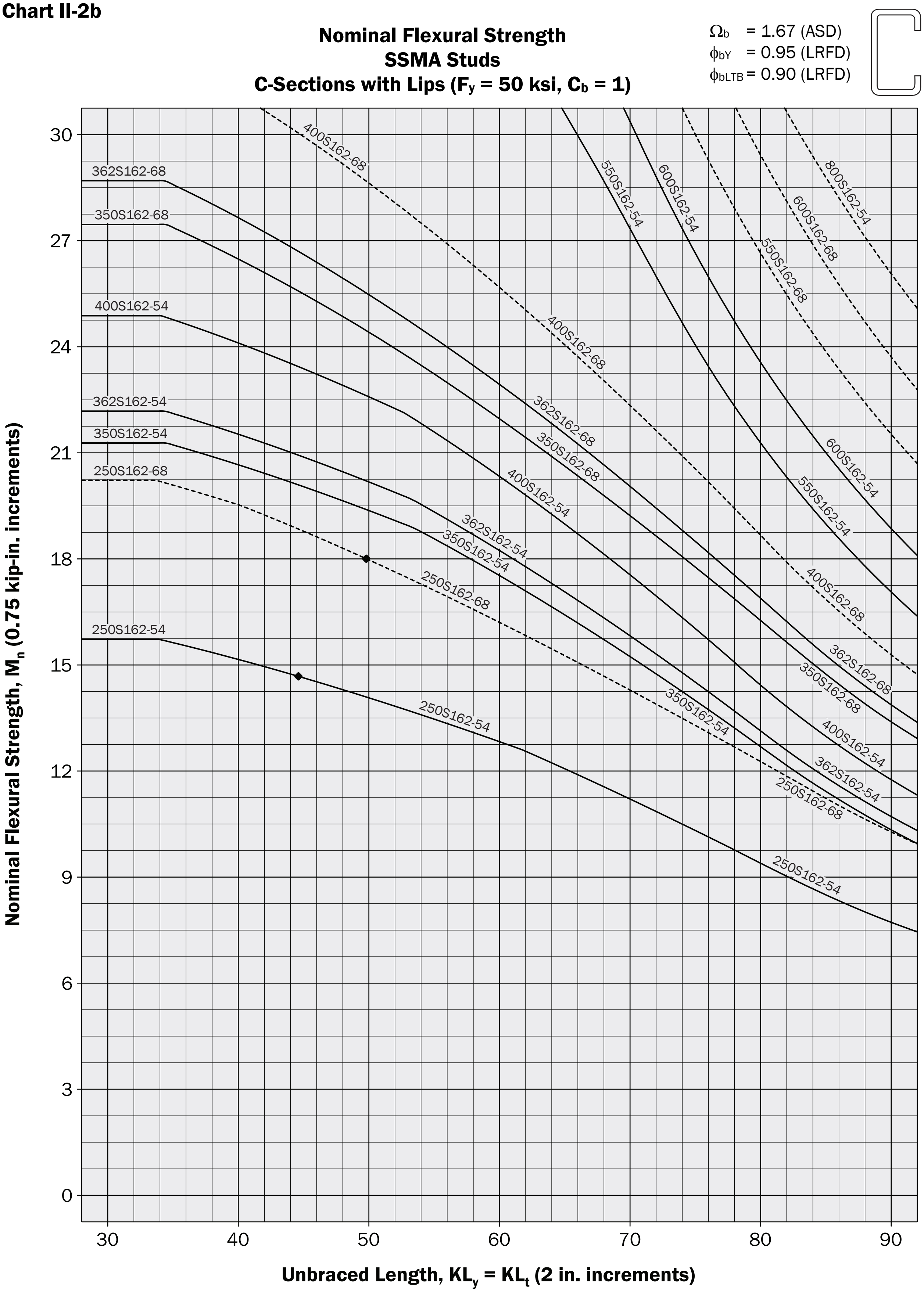


Chart II-2b

Nominal Flexural Strength
SSMA Studs
C-Sections with Lips (F_y = 50 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)

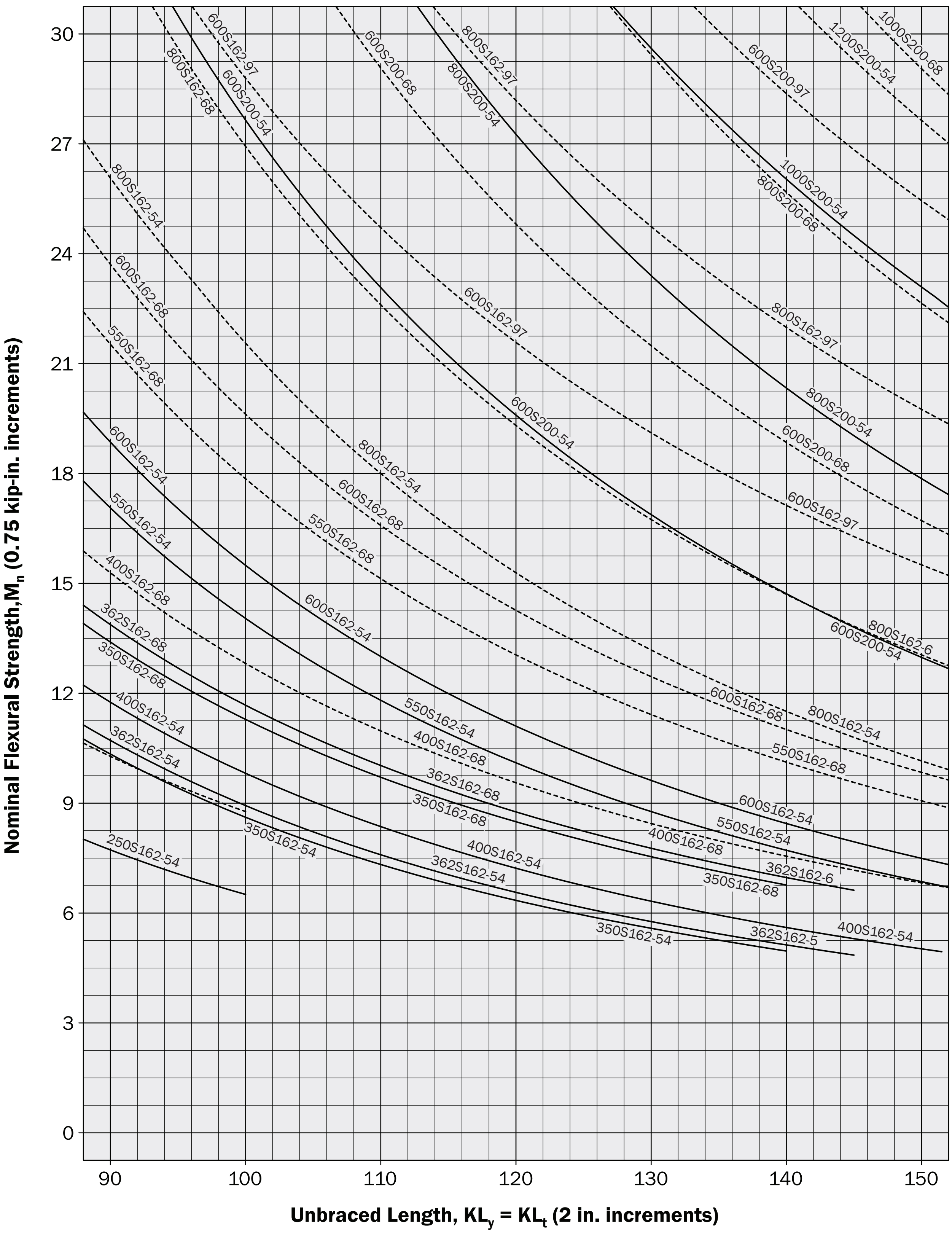


Chart II-2b

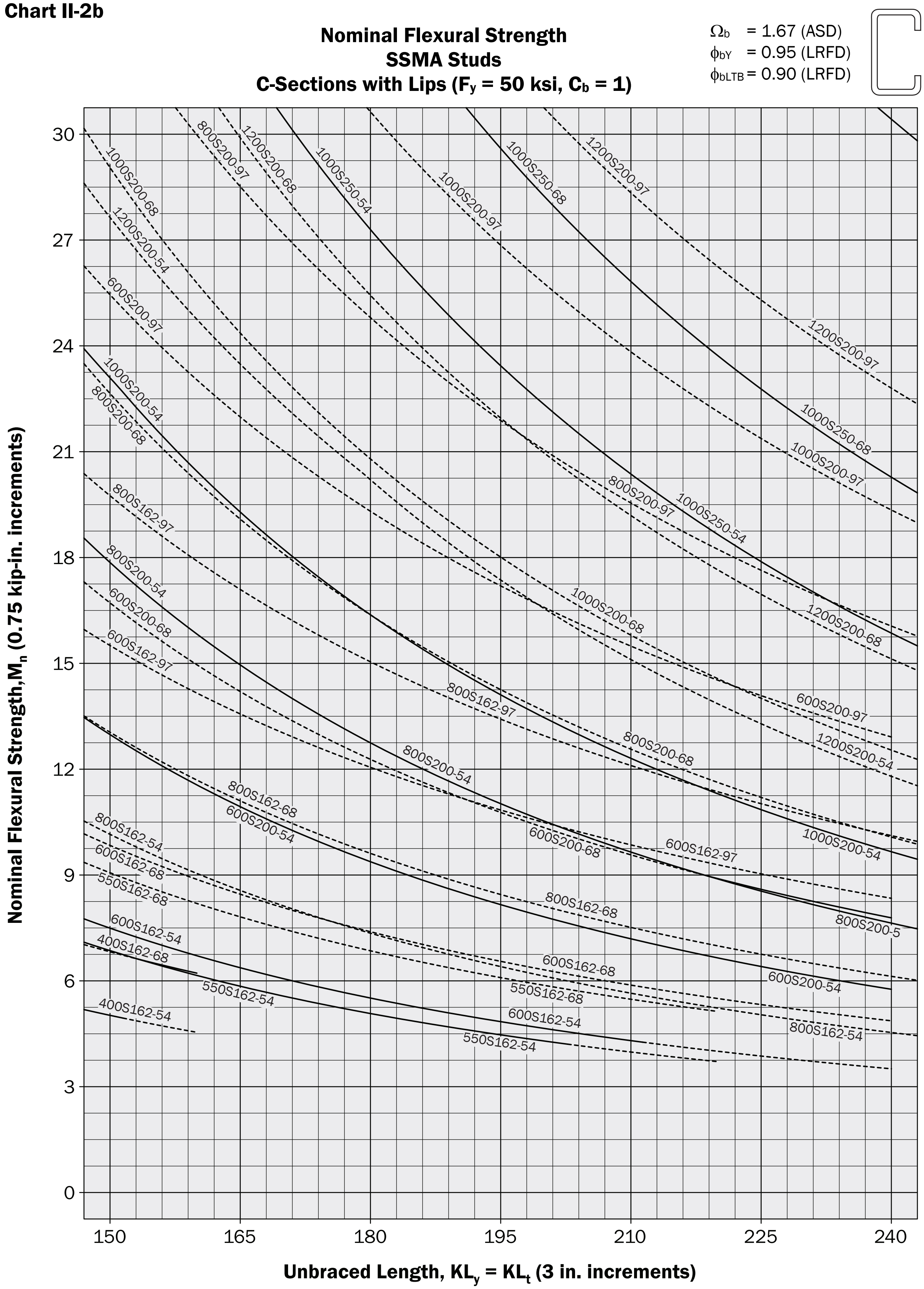

Nominal Flexural Strength
SSMA Studs
C-Sections with Lips (F_y = 50 ksi, C_b = 1)
Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)
Nominal Flexural Strength, M_n (0.75 kip-in. increments)
Unbraced Length, $KL_y = KL_t$ (3 in. increments)
0
3
6
9
12
15
18
21
24
27
30
150
165
180
195
210
225
240
1200S200-97
1000S250-68
1000S200-97
800S200-97
1000S250-54
1200S200-68
1000S200-68
1200S200-54
600S200-97
1000S200-54
800S200-68
800S162-97
800S200-54
600S200-68
600S162-97
800S162-68
600S200-54
600S162-68
550S162-68
600S162-54
400S162-68
550S162-54
400S162-54
800S162-54

Chart II-2b

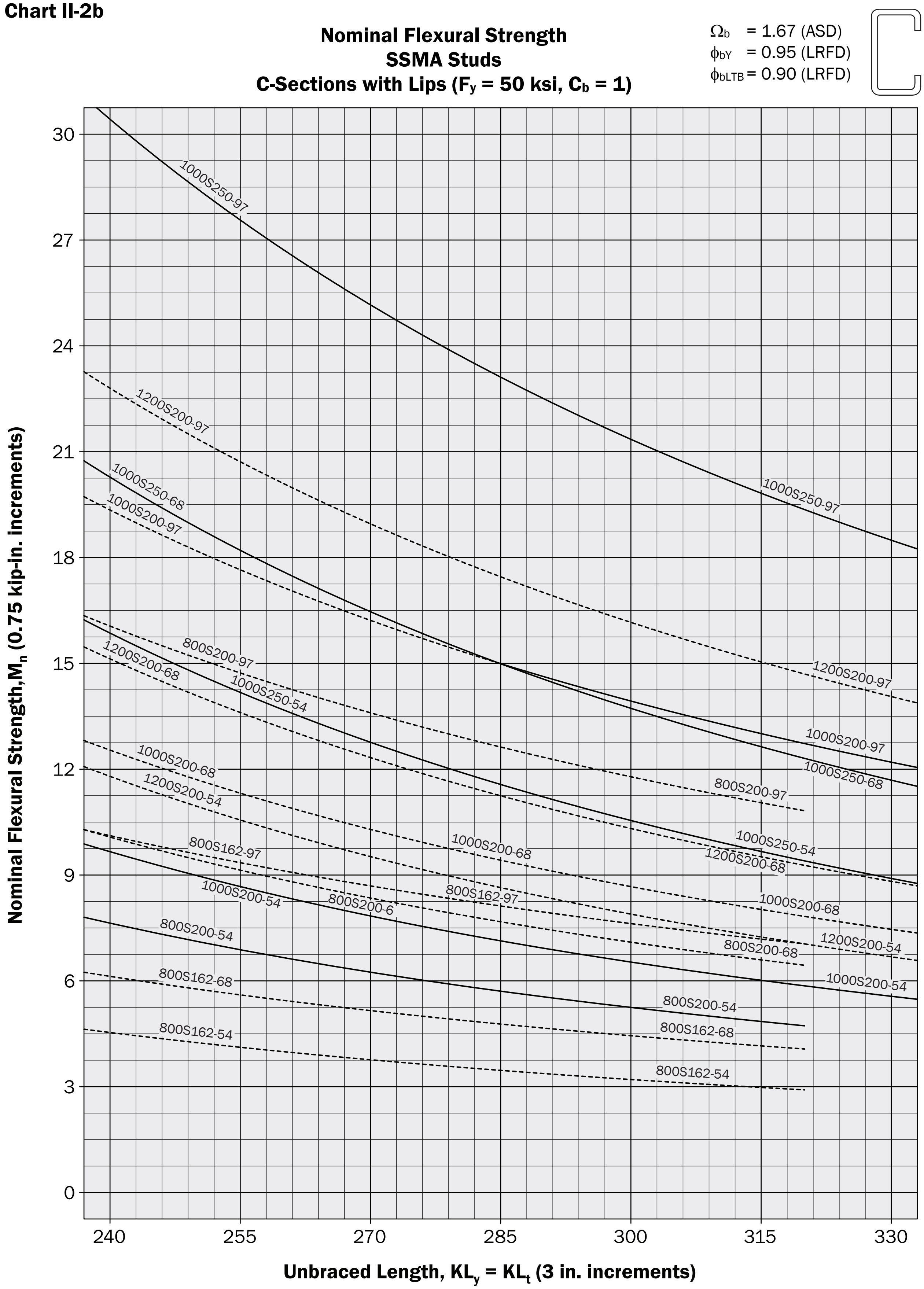
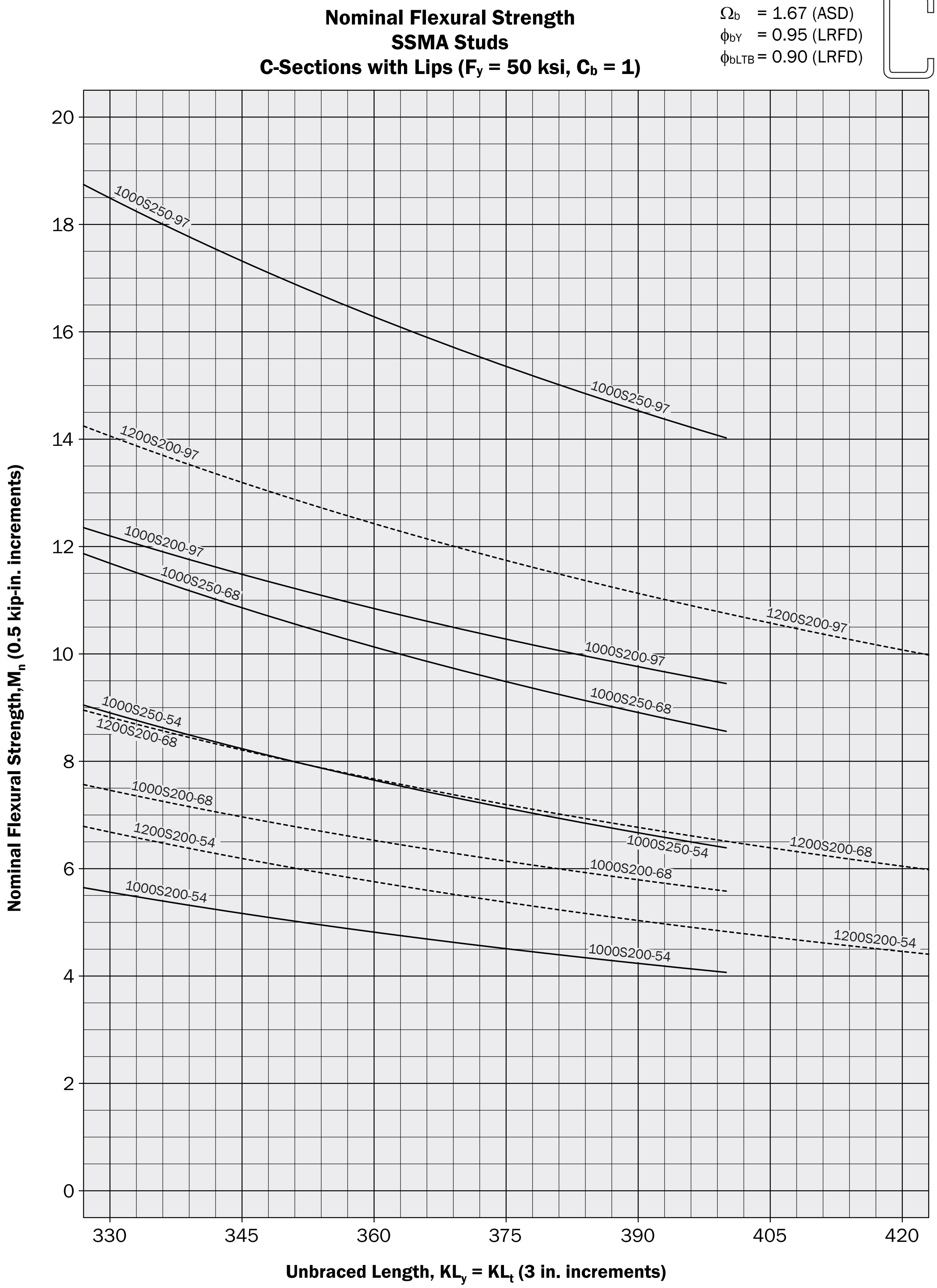
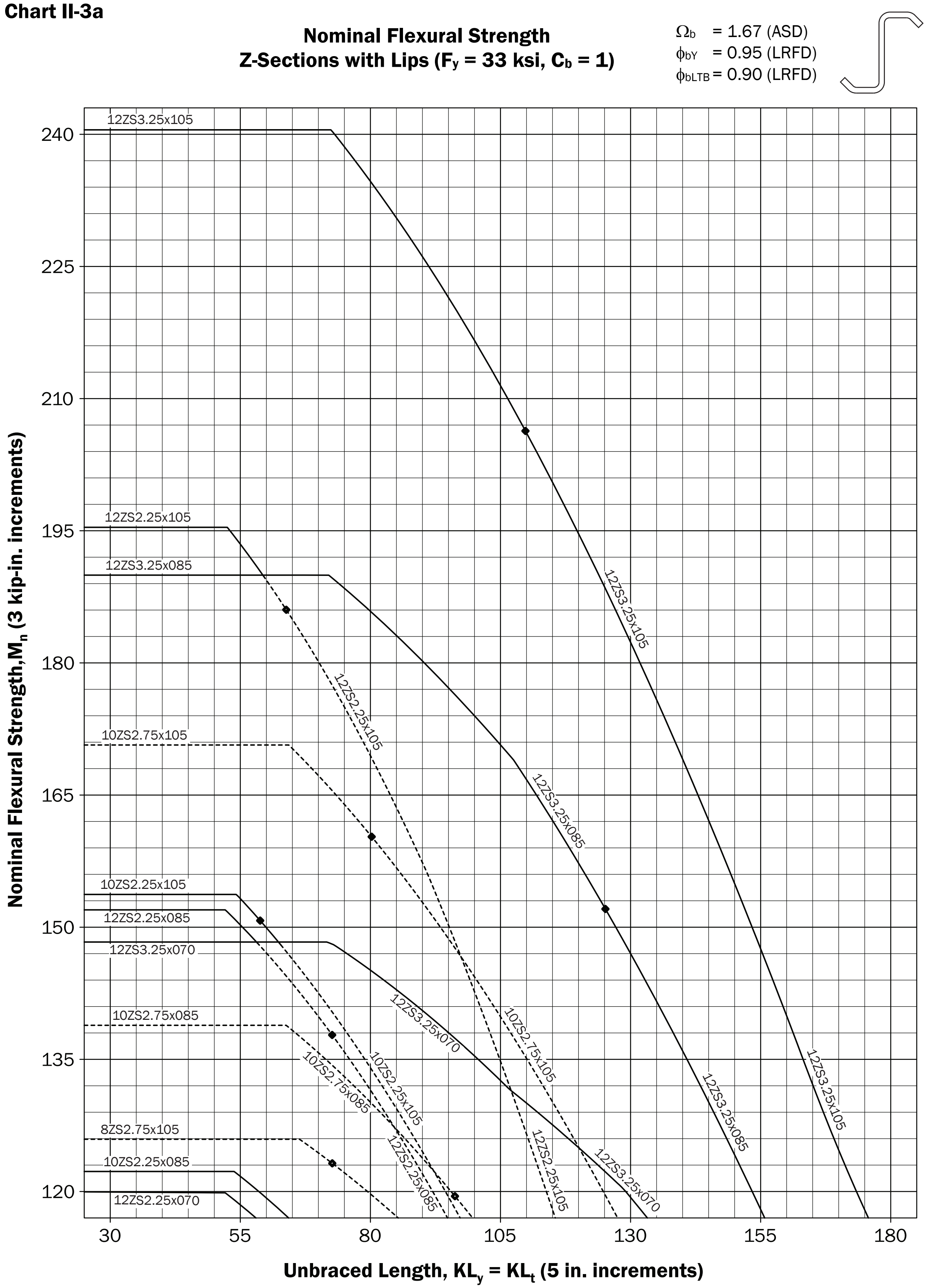

Nominal Flexural Strength
SSMA Studs
C-Sections with Lips (F_y = 50 ksi, C_b = 1)
Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)
Nominal Flexural Strength, M_n (0.75 kip-in. increments)
Unbraced Length, $KL_y = KL_t$ (3 in. increments)
0
3
6
9
12
15
18
21
24
27
30
240
255
270
285
300
315
330
1000S250-97
1200S200-97
1000S250-68
1000S200-97
800S200-97
1200S200-68
1000S250-54
1000S200-68
1200S200-54
800S162-97
1000S200-54
800S200-6
800S200-54
800S162-68
800S162-54
800S200-68
1200S200-54

Chart II-2b



Nominal Flexural Strength
SSMA Studs
C-Sections with Lips (F_y = 50 ksi, C_b = 1)
Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)
Nominal Flexural Strength, M_n (0.5 kip-in. increments)
Unbraced Length, $KL_y = KL_t$ (3 in. increments)
0
2
4
6
8
10
12
14
16
18
20
330
345
360
375
390
405
420
1000S250-97
1200S200-97
1000S200-97
1000S250-68
1000S250-54
1200S200-68
1000S200-68
1200S200-54
1000S200-54

Chart II-3a

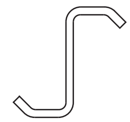


Nominal Flexural Strength
Z-Sections with Lips (F_y = 33 ksi, C_b = 1)
Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)
Nominal Flexural Strength, M_n (3 kip-in. increments)
Unbraced Length, $KL_y = KL_t$ (5 in. increments)
240
225
210
195
180
165
150
135
120
30
55
80
105
130
155
180
12ZS3.25x105
12ZS2.25x105
12ZS3.25x085
10ZS2.75x105
10ZS2.25x105
12ZS2.25x085
12ZS3.25x070
10ZS2.75x085
8ZS2.75x105
10ZS2.25x085
12ZS2.25x070

Chart II-3a

Nominal Flexural Strength
Z-Sections with Lips (F_y = 33 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)



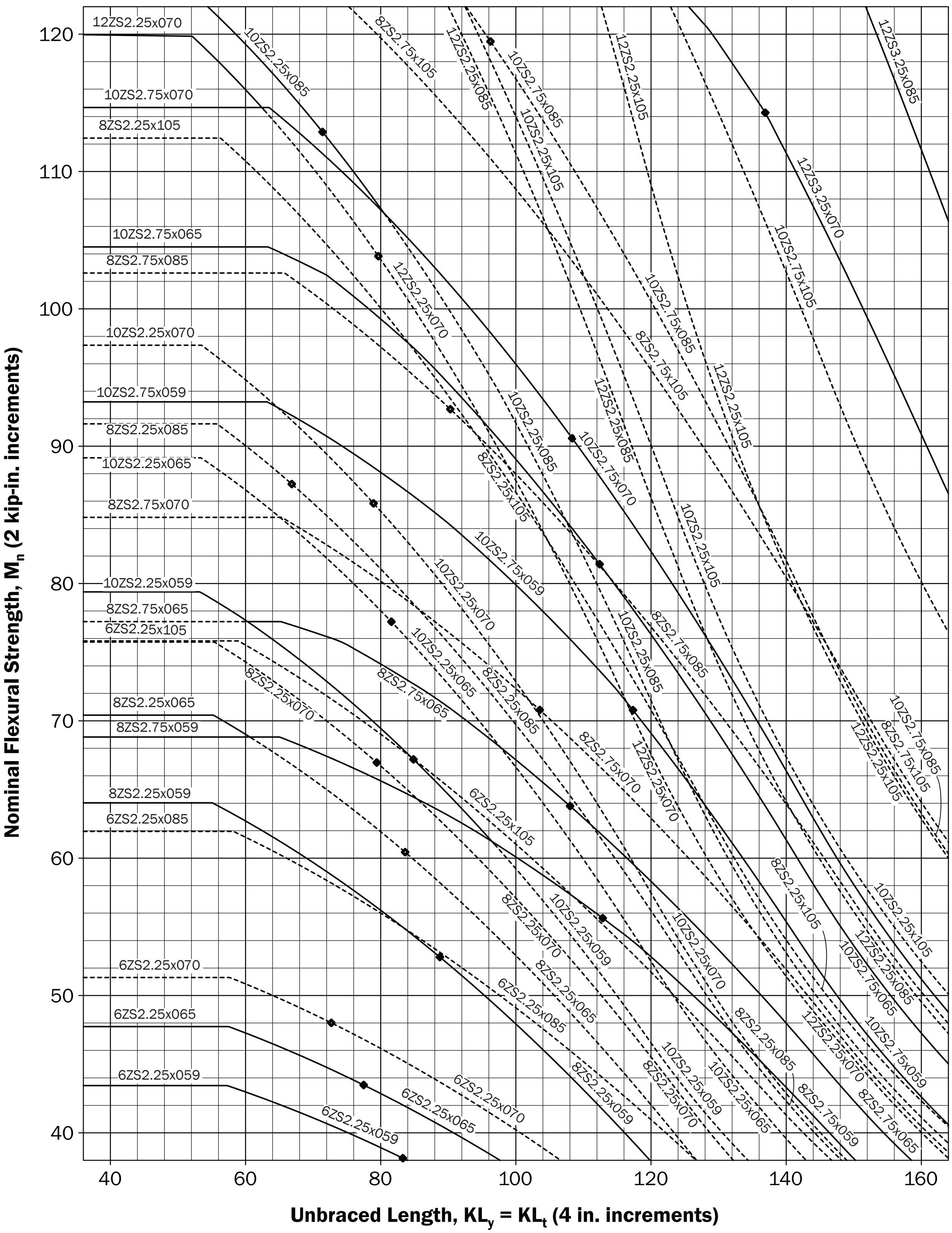
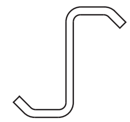


Chart II-3a

Nominal Flexural Strength
Z-Sections with Lips (F_y = 33 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)



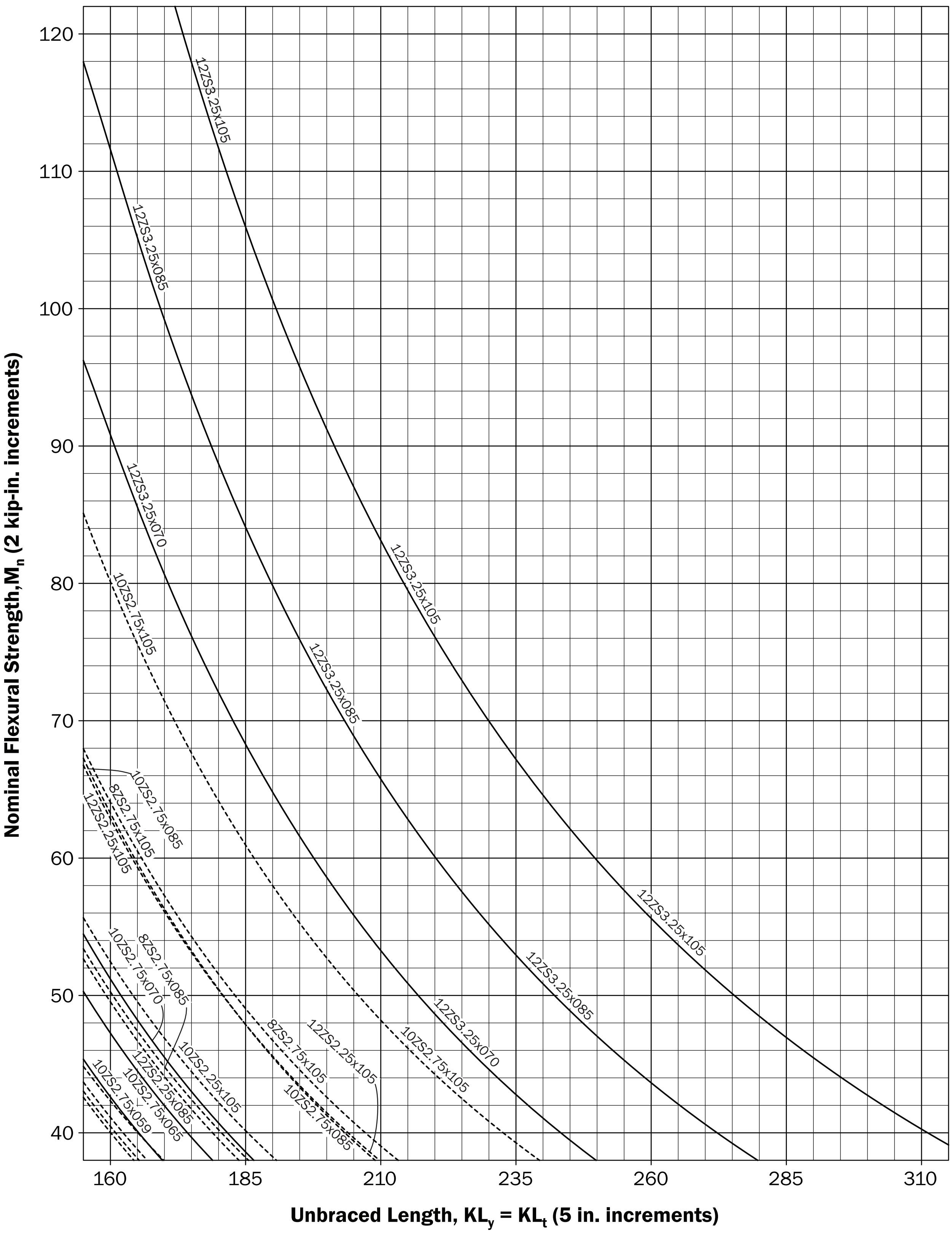


Chart II-3a

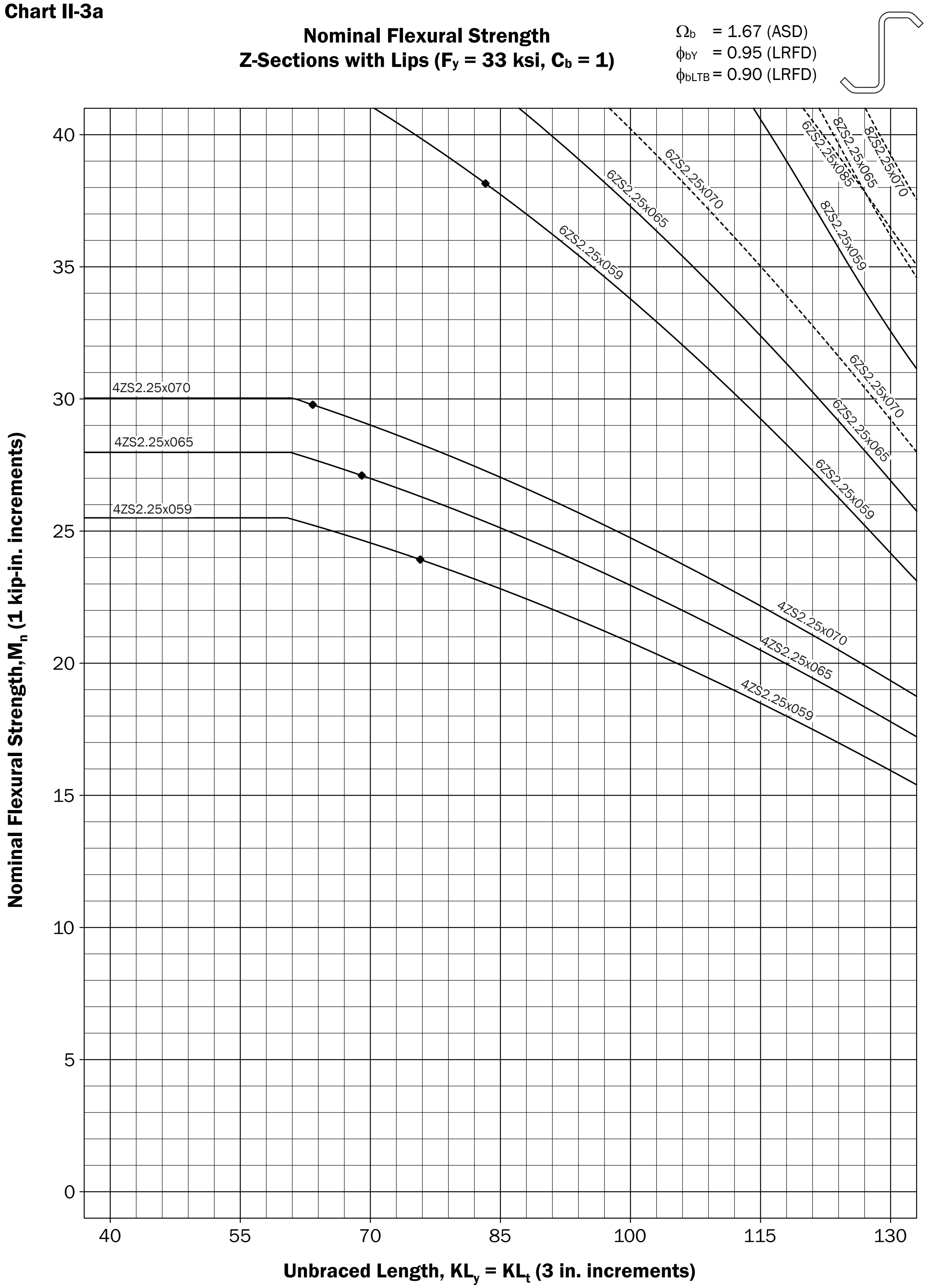
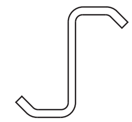

Nominal Flexural Strength
Z-Sections with Lips (F_y = 33 ksi, C_b = 1)
Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)
4ZS2.25x070
4ZS2.25x065
4ZS2.25x059
6ZS2.25x059
6ZS2.25x065
6ZS2.25x070
6ZS2.25x085
8ZS2.25x059
8ZS2.25x065
8ZS2.25x070
Nominal Flexural Strength, M_n (1 kip-in. increments)
Unbraced Length, $KL_y = KL_t$ (3 in. increments)
0
5
10
15
20
25
30
35
40
40
55
70
85
100
115
130

Chart II-3a

Nominal Flexural Strength
Z-Sections with Lips (F_y = 33 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)



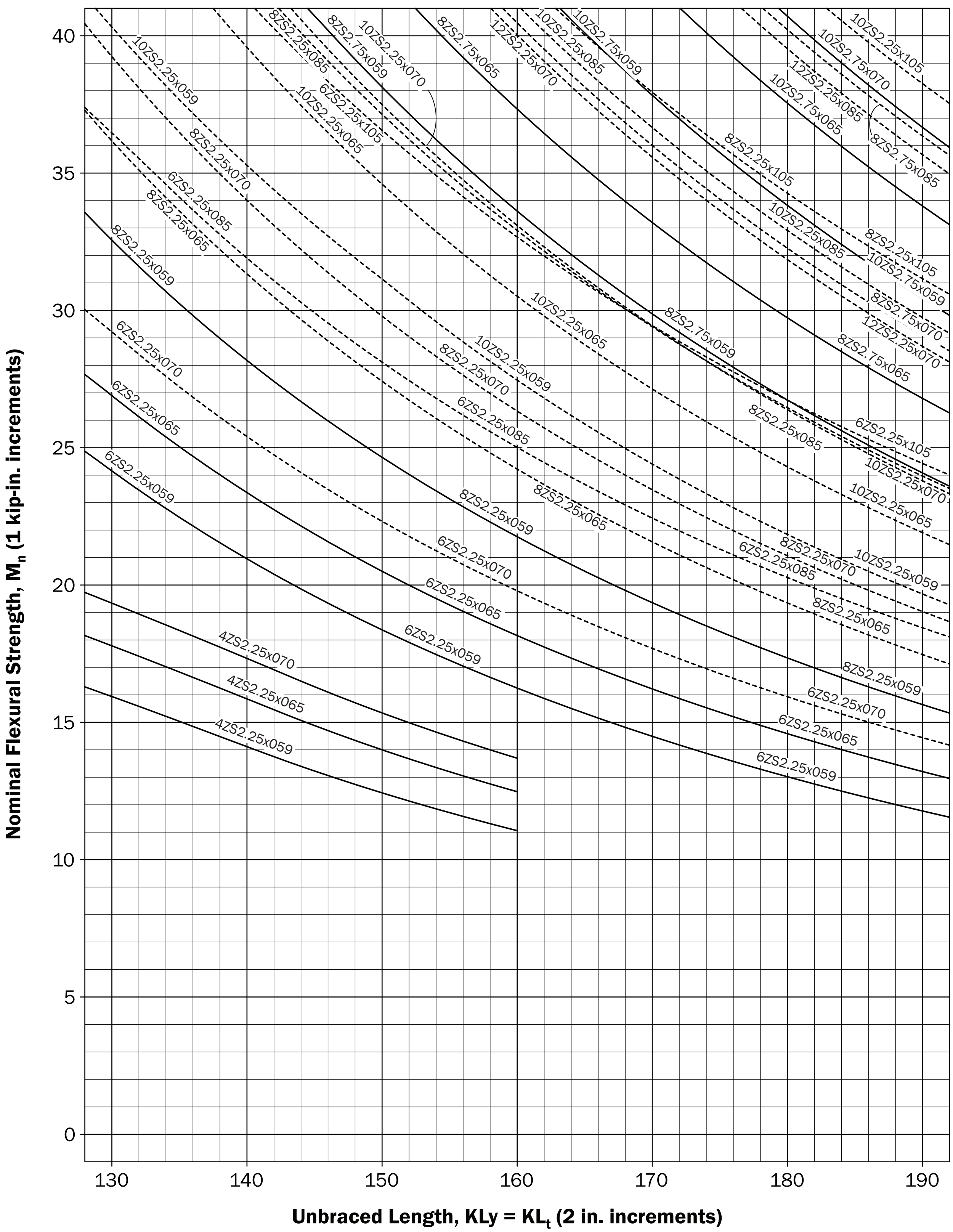


Chart II-3a

Nominal Flexural Strength
Z-Sections with Lips (F_y = 33 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)

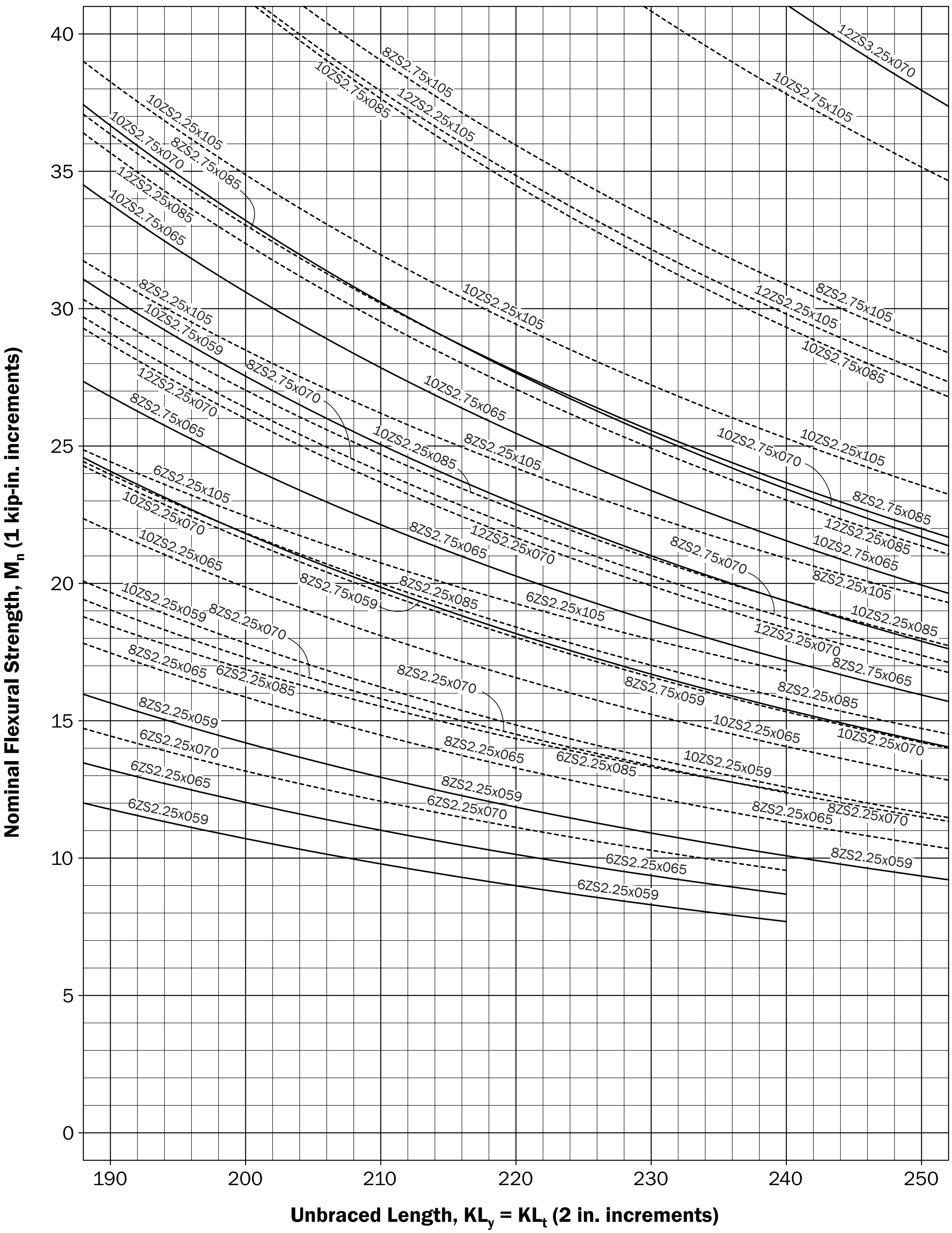


Chart II-3a

Nominal Flexural Strength
Z-Sections with Lips (F_y = 33 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)

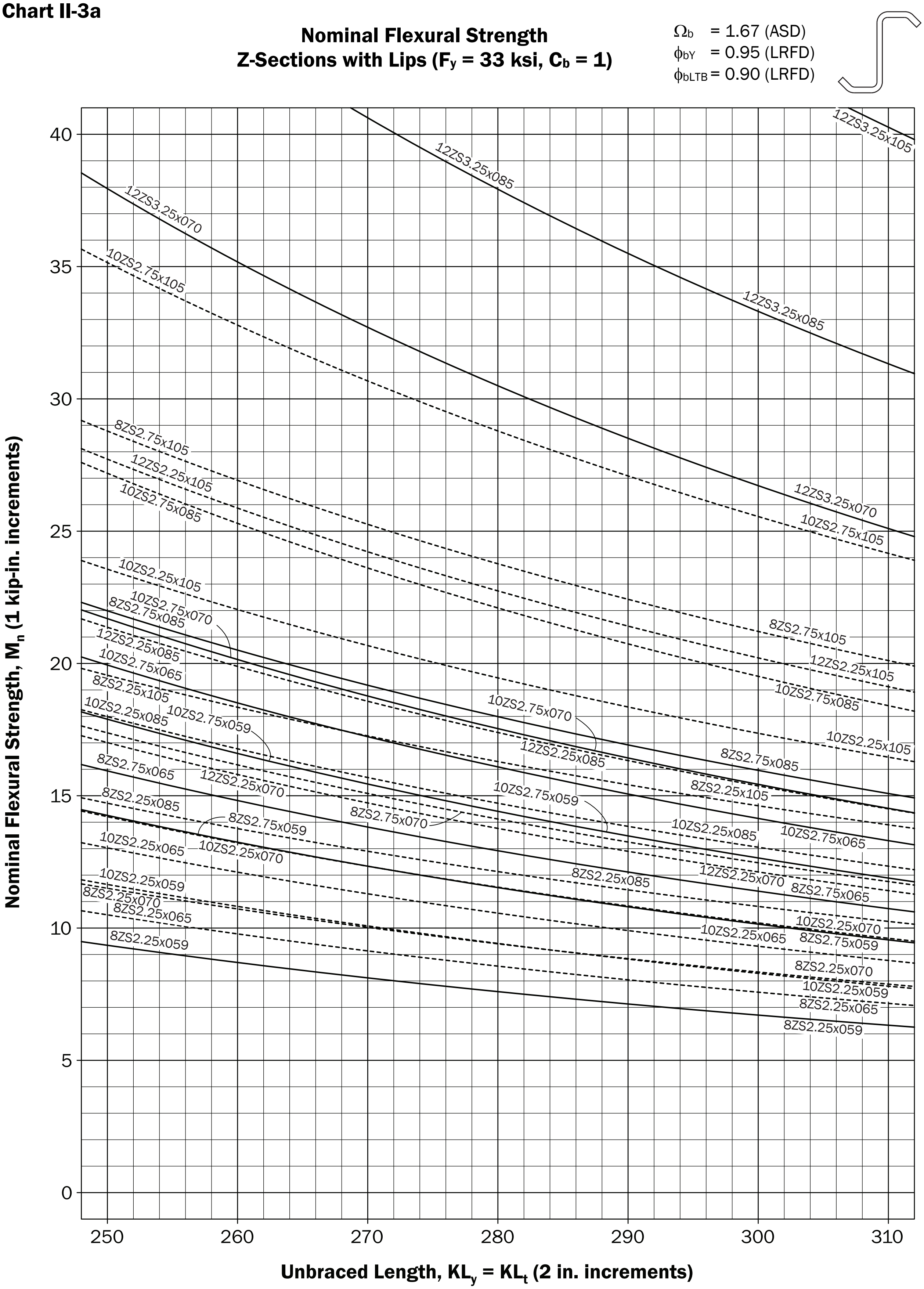
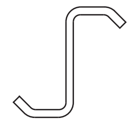


Chart II-3a

Nominal Flexural Strength
Z-Sections with Lips (F_y = 33 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)



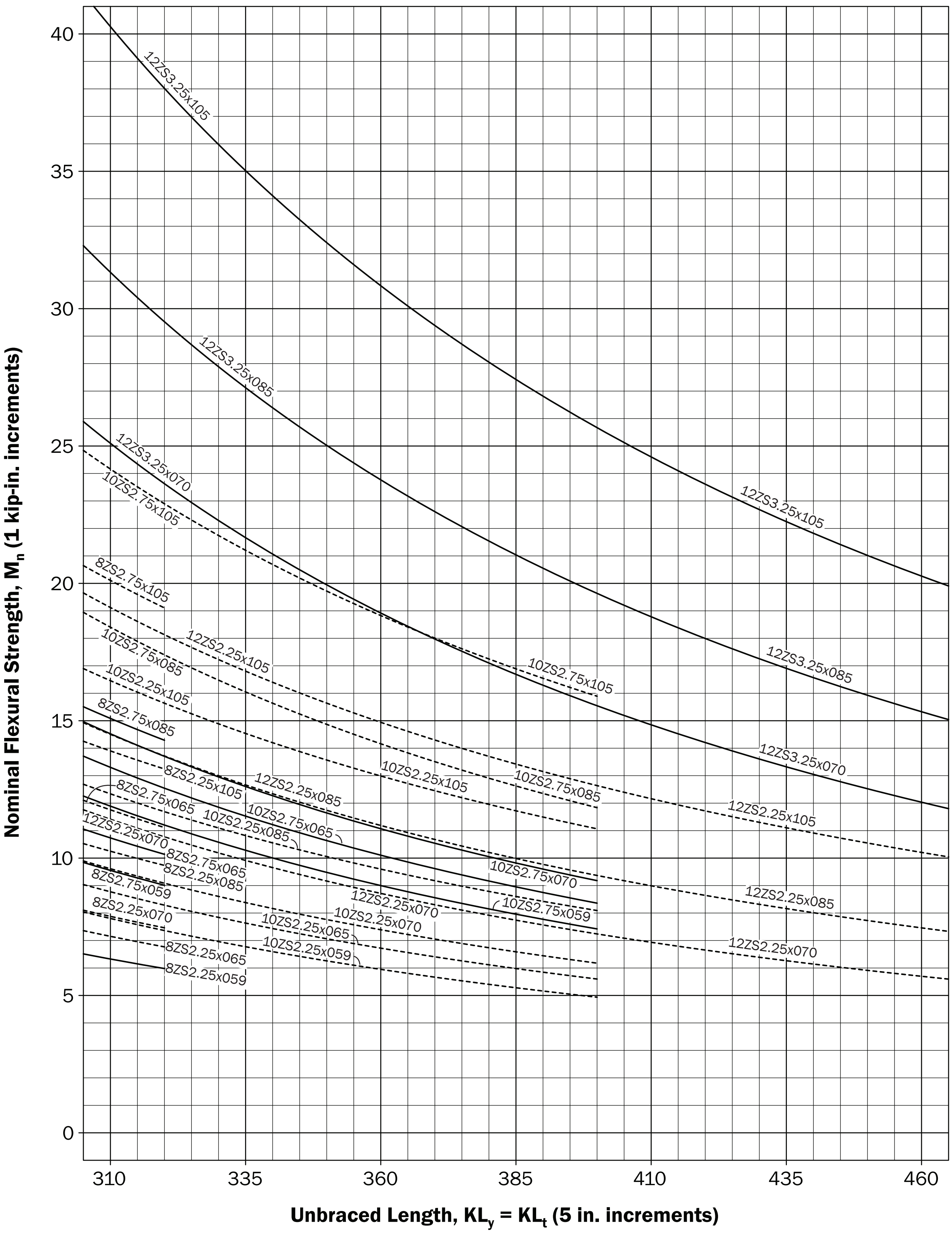


Chart II-3b

Nominal Flexural Strength
Z-Sections with Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)

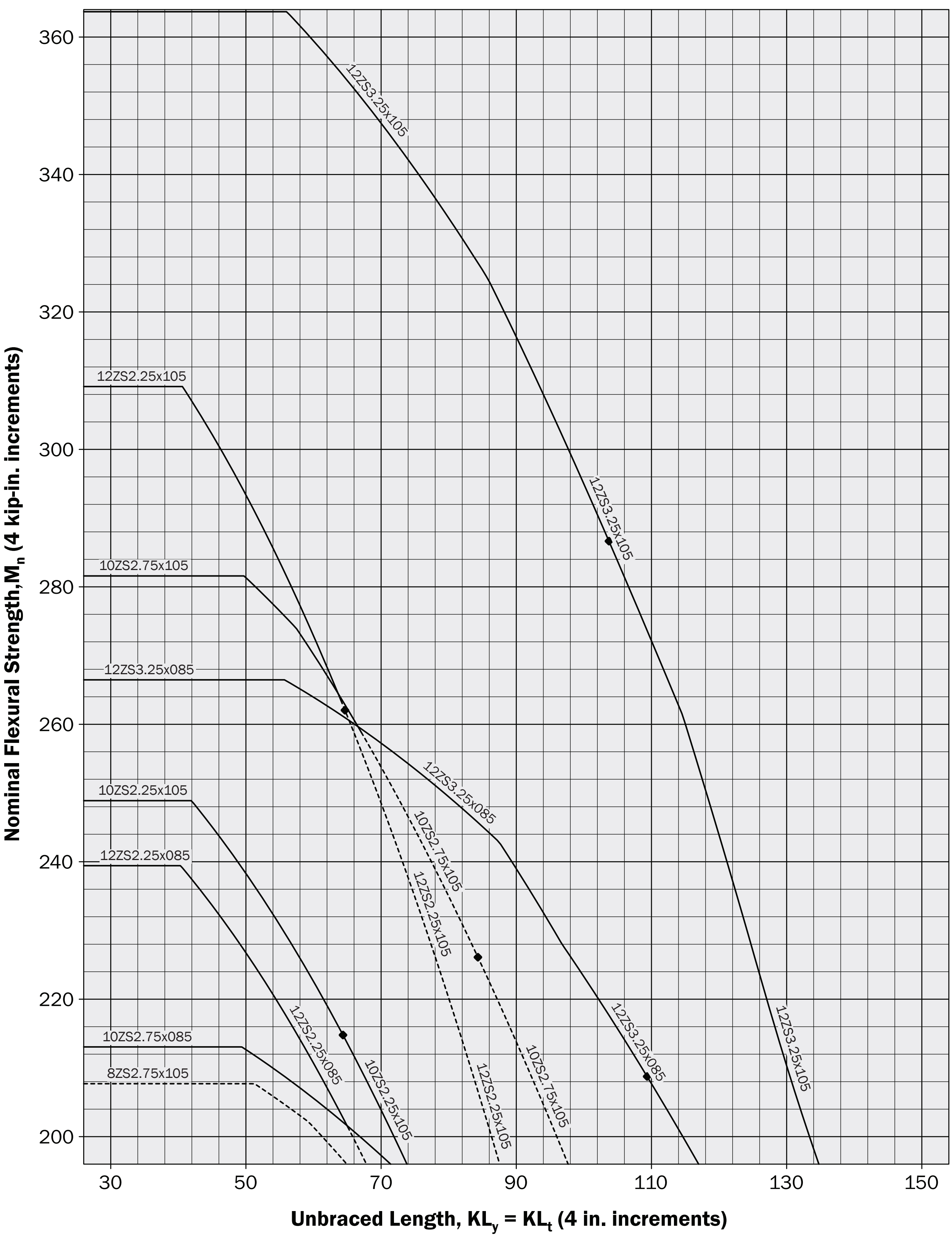


Chart II-3b

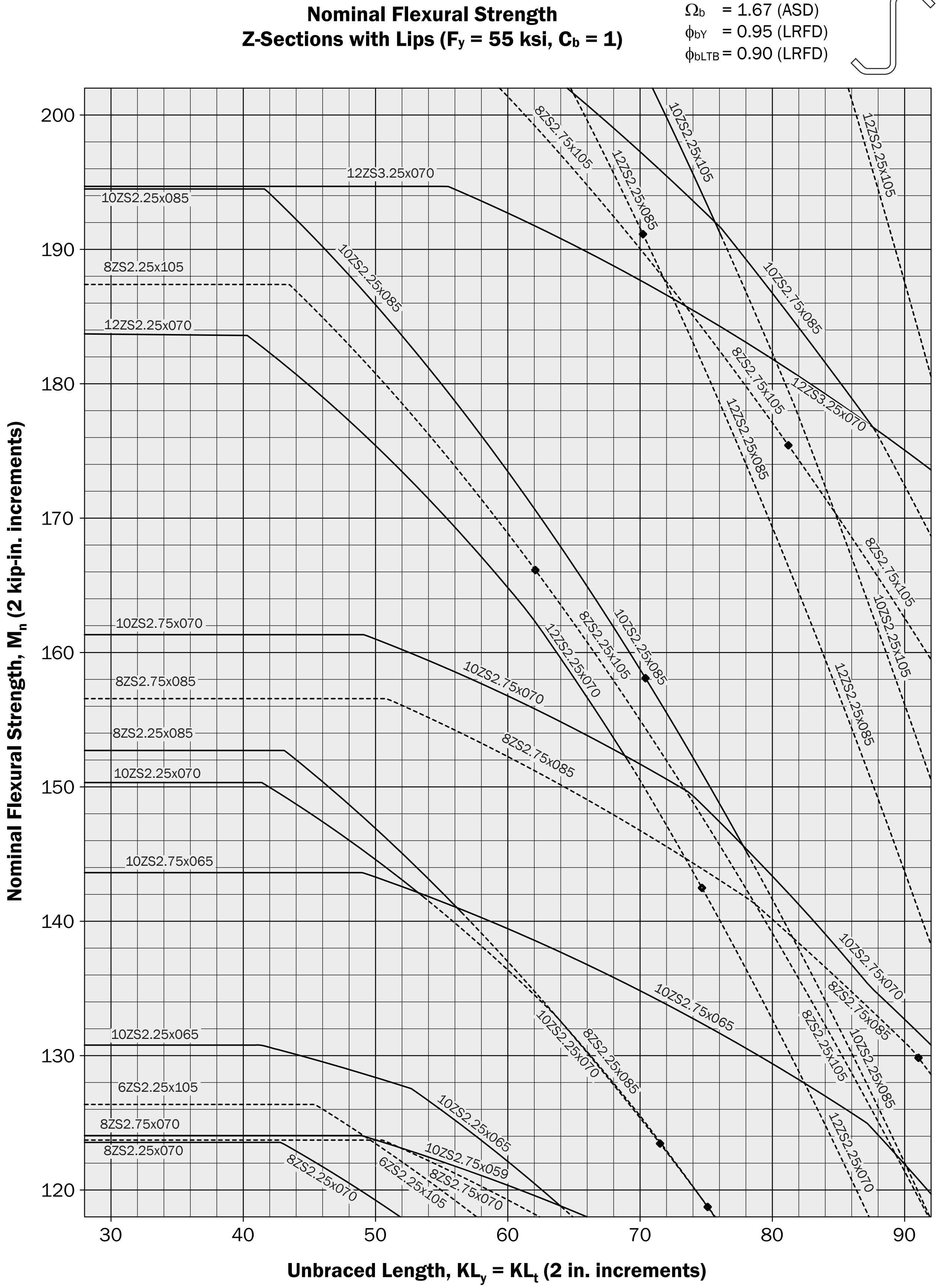

Nominal Flexural Strength
Z-Sections with Lips (F_y = 55 ksi, C_b = 1)
Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)
Nominal Flexural Strength, M_n (2 kip-in. increments)
Unbraced Length, $KL_y = KL_t$ (2 in. increments)
200
190
180
170
160
150
140
130
120
30
40
50
60
70
80
90
12ZS3.25x070
10ZS2.25x085
8ZS2.25x105
12ZS2.25x070
10ZS2.75x070
8ZS2.75x085
8ZS2.25x085
10ZS2.25x070
10ZS2.75x065
10ZS2.25x065
6ZS2.25x105
8ZS2.75x070
8ZS2.25x070
8ZS2.75x105
12ZS2.25x085
10ZS2.25x105
12ZS2.25x105
10ZS2.75x085
10ZS2.75x059

Chart II-3b

Nominal Flexural Strength
Z-Sections with Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)

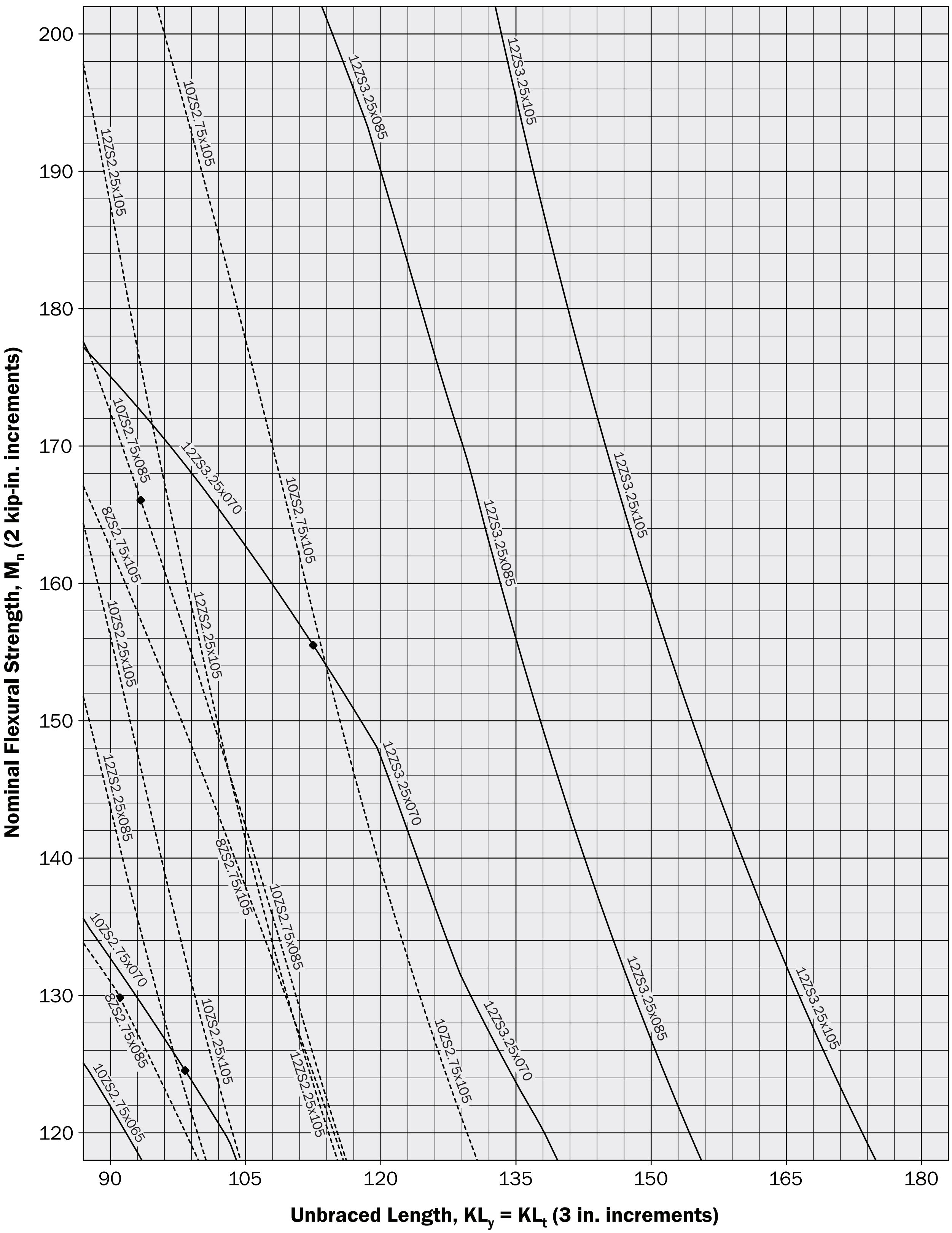
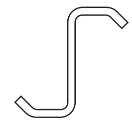


Chart II-3b

Nominal Flexural Strength
Z-Sections with Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)



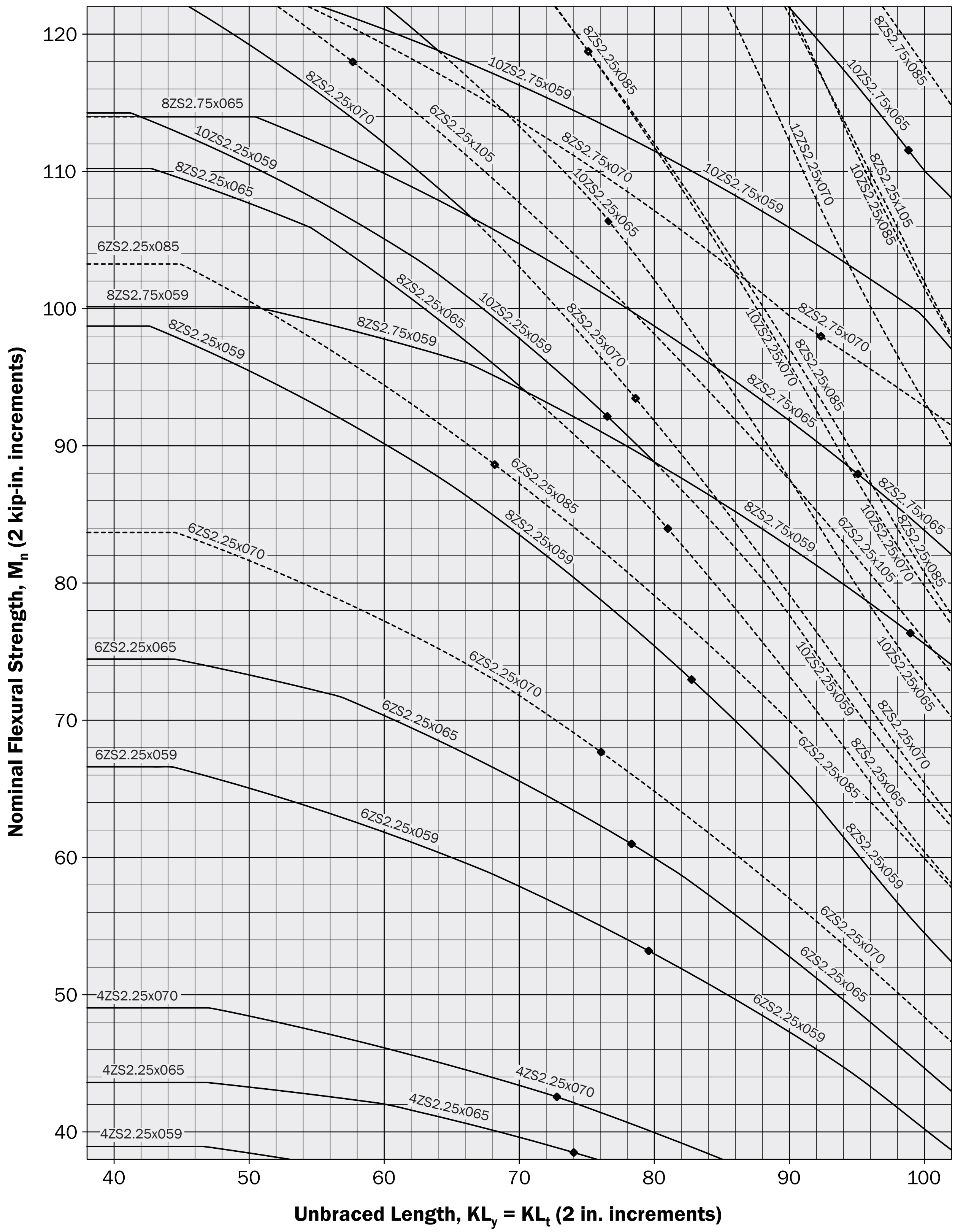


Chart II-3b

Nominal Flexural Strength
Z-Sections with Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)

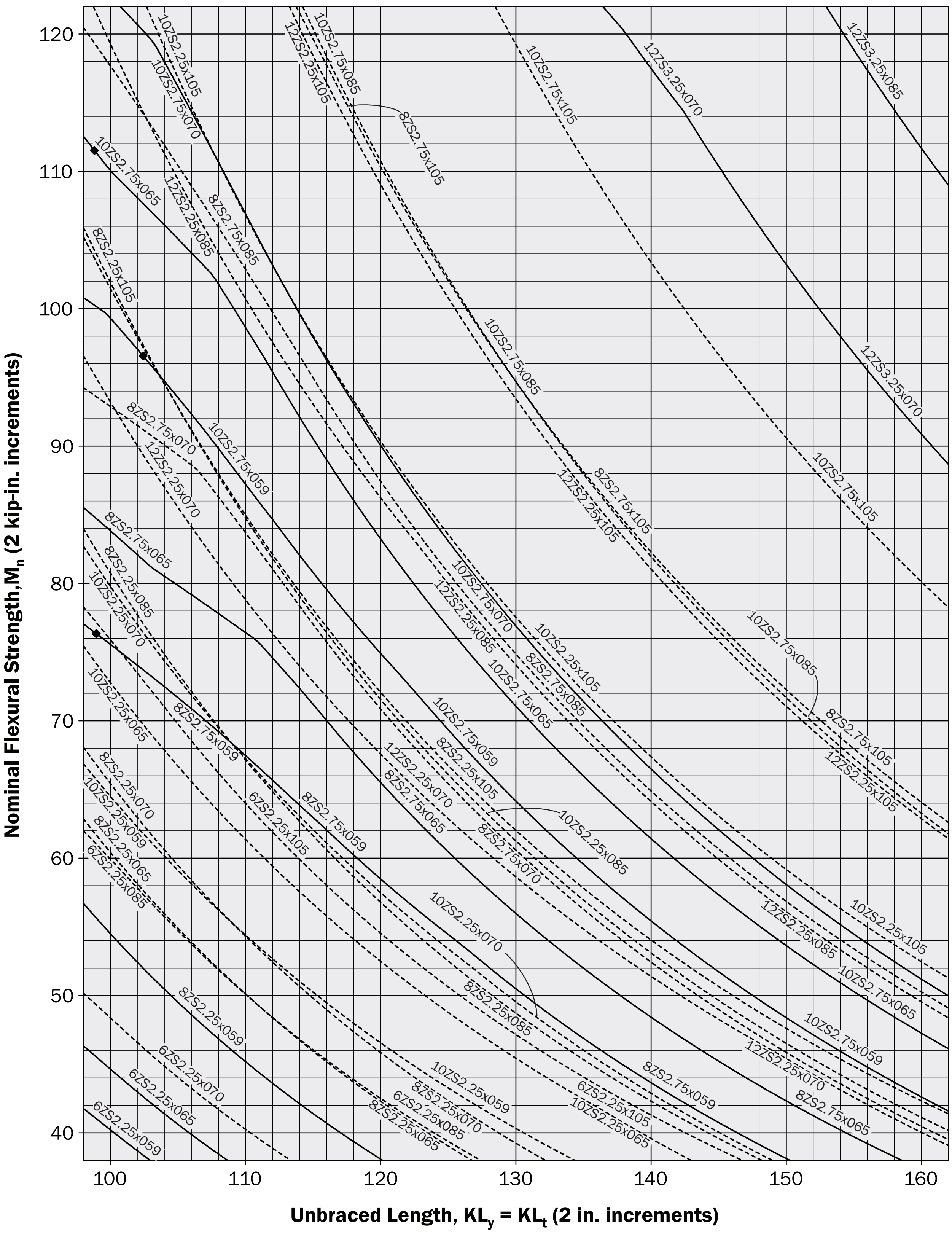


Chart II-3b

Nominal Flexural Strength
Z-Sections with Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)

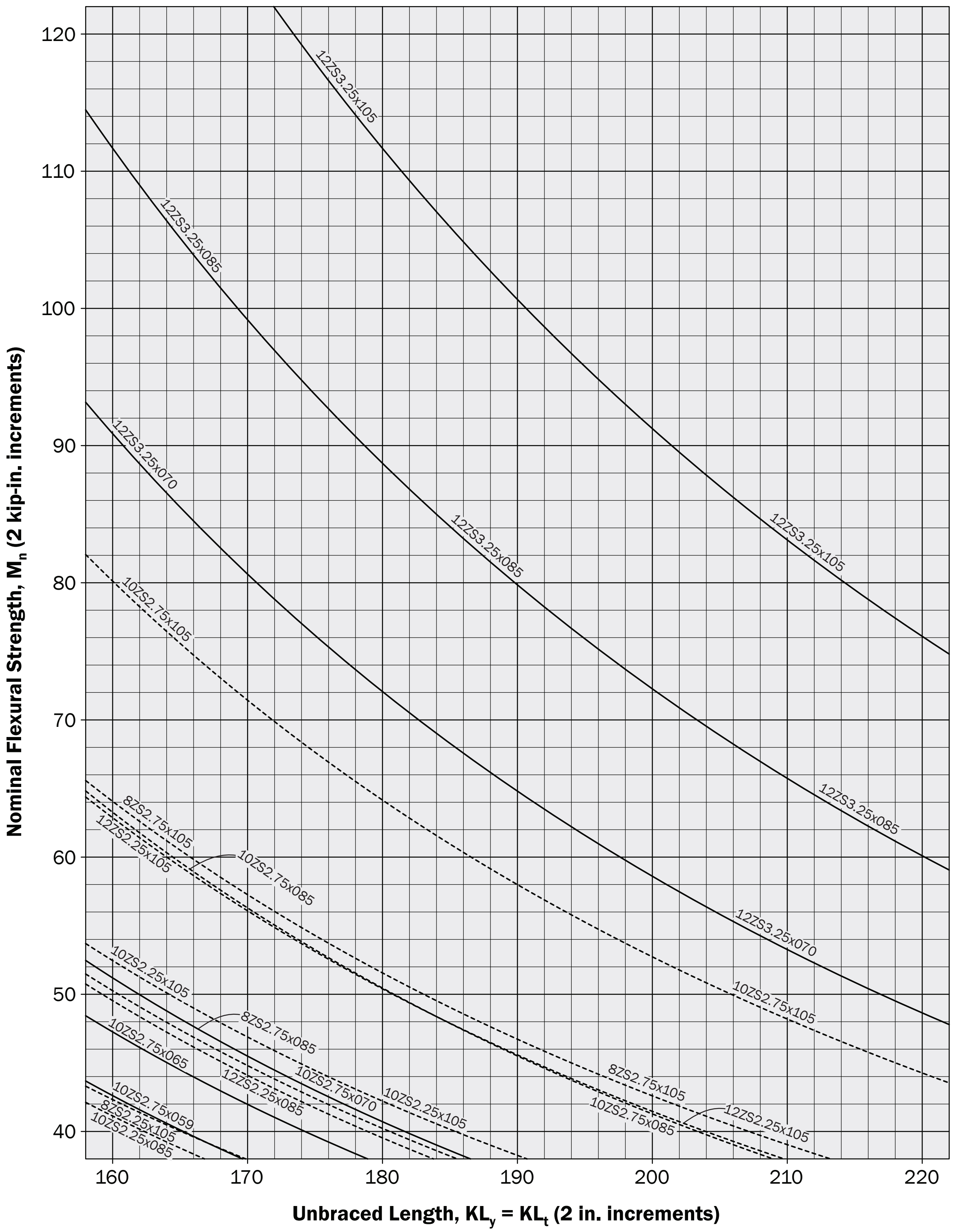


Chart II-3b

Nominal Flexural Strength
Z-Sections with Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)

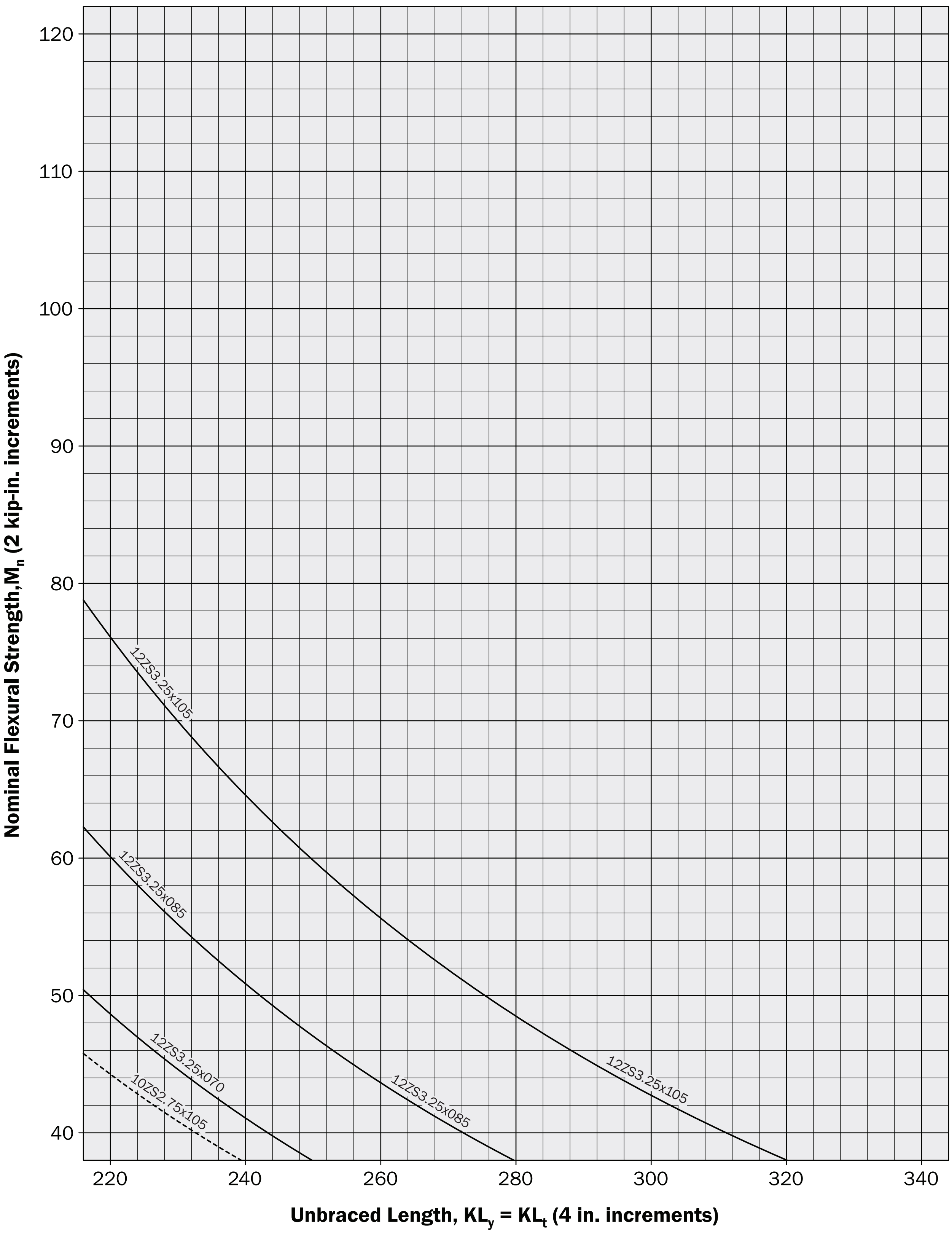


Chart II-3b

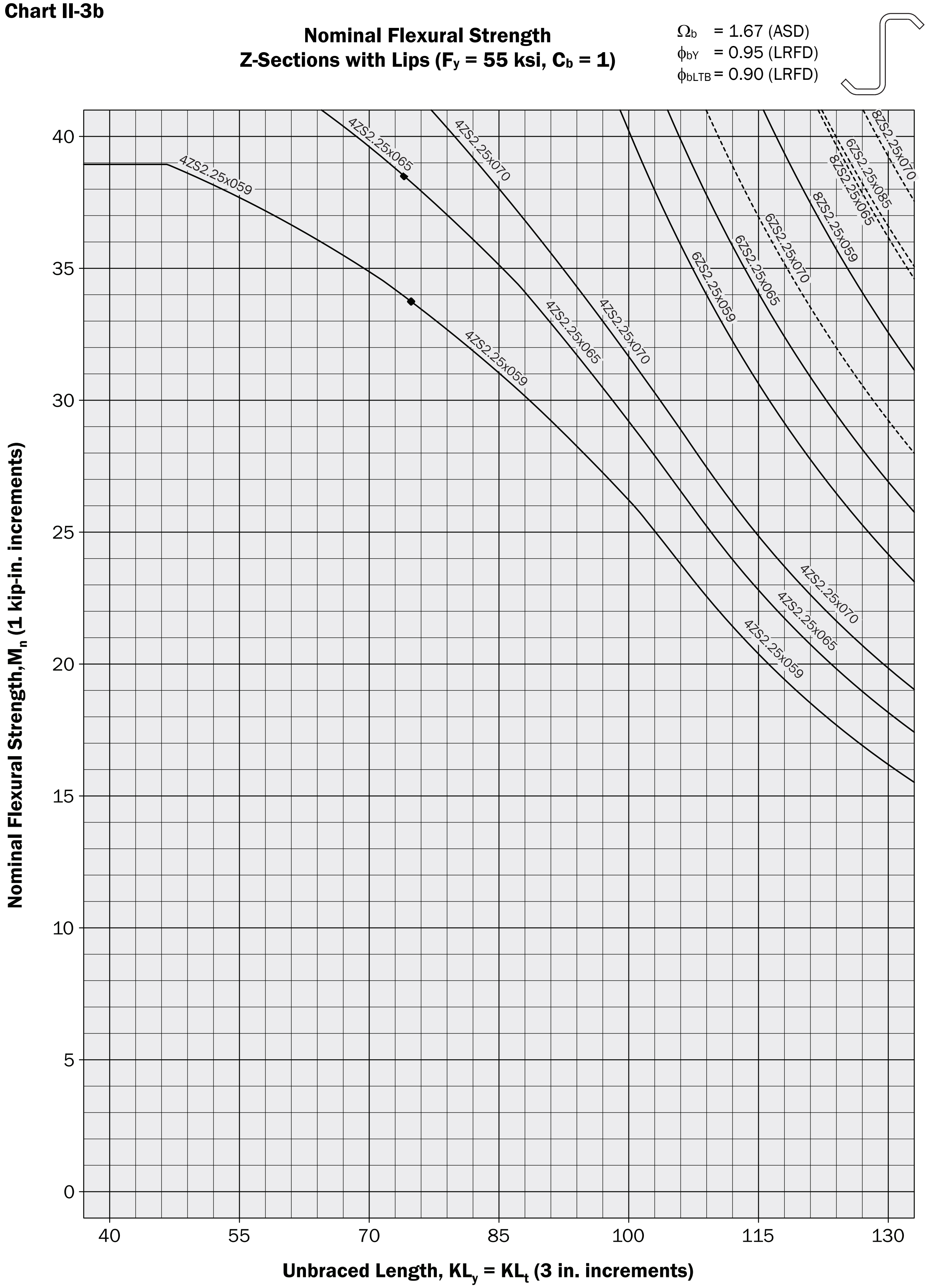
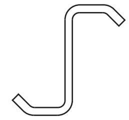

Nominal Flexural Strength
Z-Sections with Lips (F_y = 55 ksi, C_b = 1)
Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)
Nominal Flexural Strength, M_n (1 kip-in. increments)
Unbraced Length, $KL_y = KL_t$ (3 in. increments)
40
35
30
25
20
15
10
5
0
40
55
70
85
100
115
130
4ZS2.25x059
4ZS2.25x065
4ZS2.25x070
6ZS2.25x059
6ZS2.25x065
6ZS2.25x070
8ZS2.25x059
8ZS2.25x065
6ZS2.25x085
8ZS2.25x070

Chart II-3b

Nominal Flexural Strength
Z-Sections with Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)



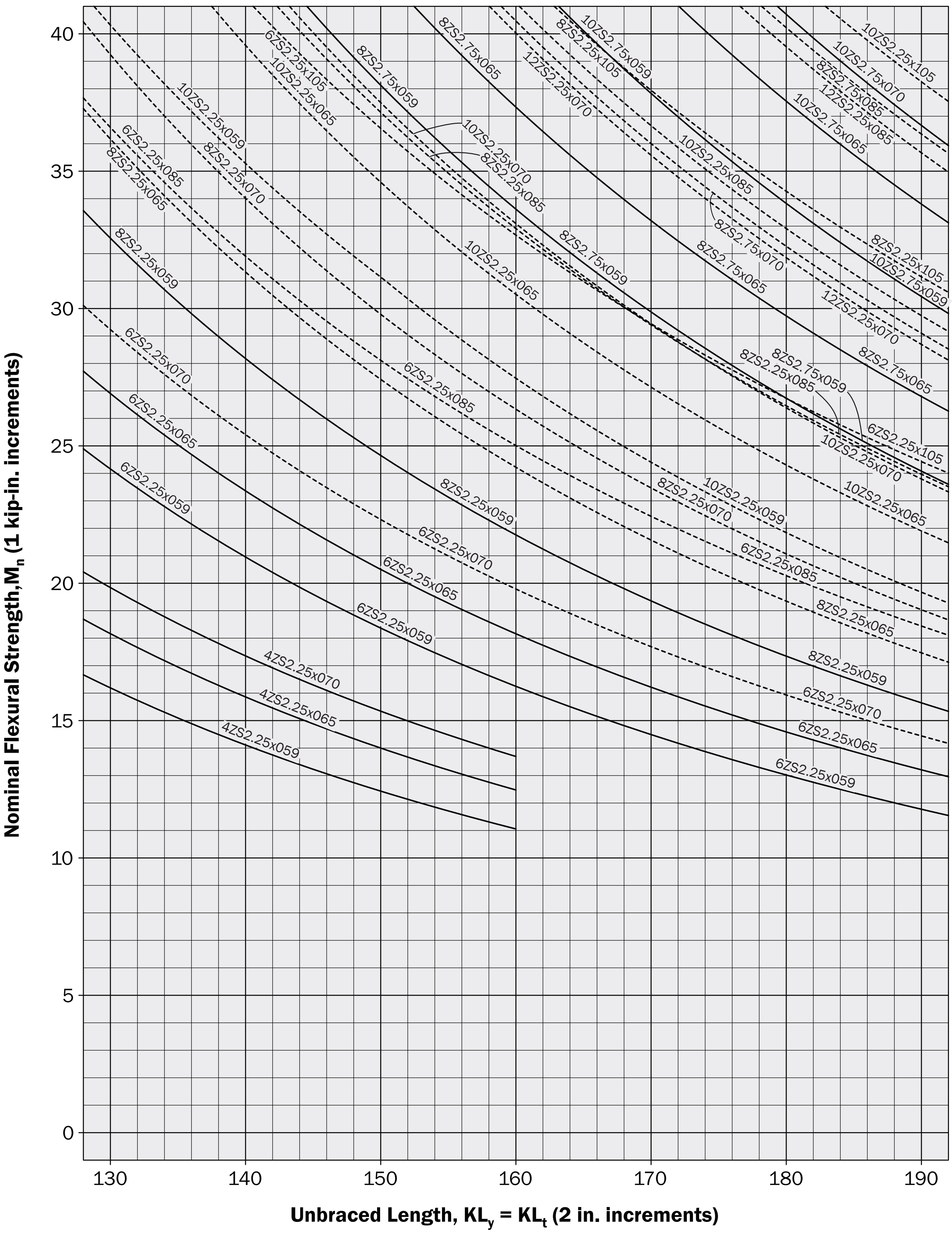
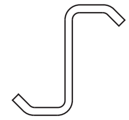


Chart II-3b

Nominal Flexural Strength
Z-Sections with Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)



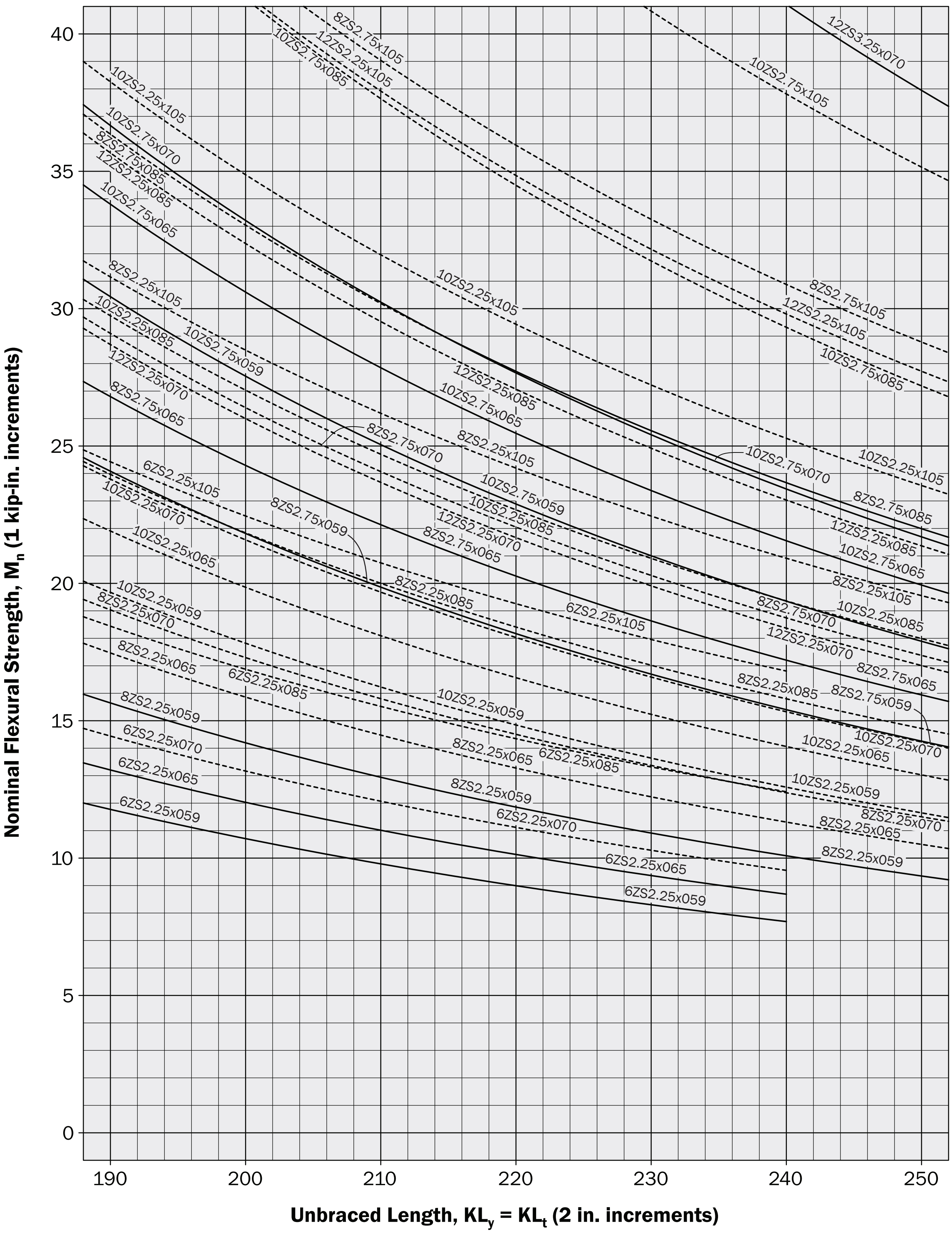
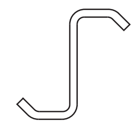


Chart II-3b

Nominal Flexural Strength
Z-Sections with Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)



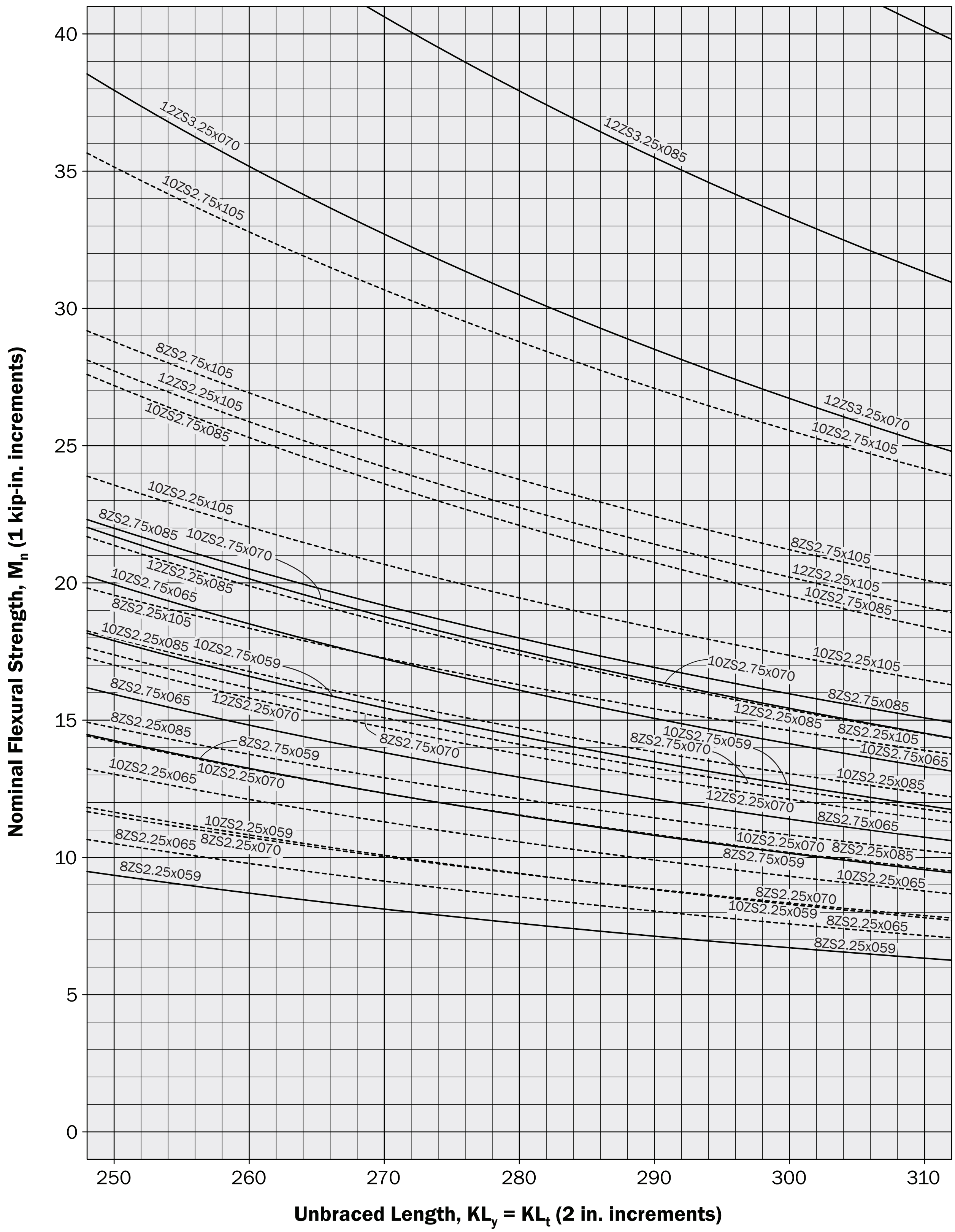
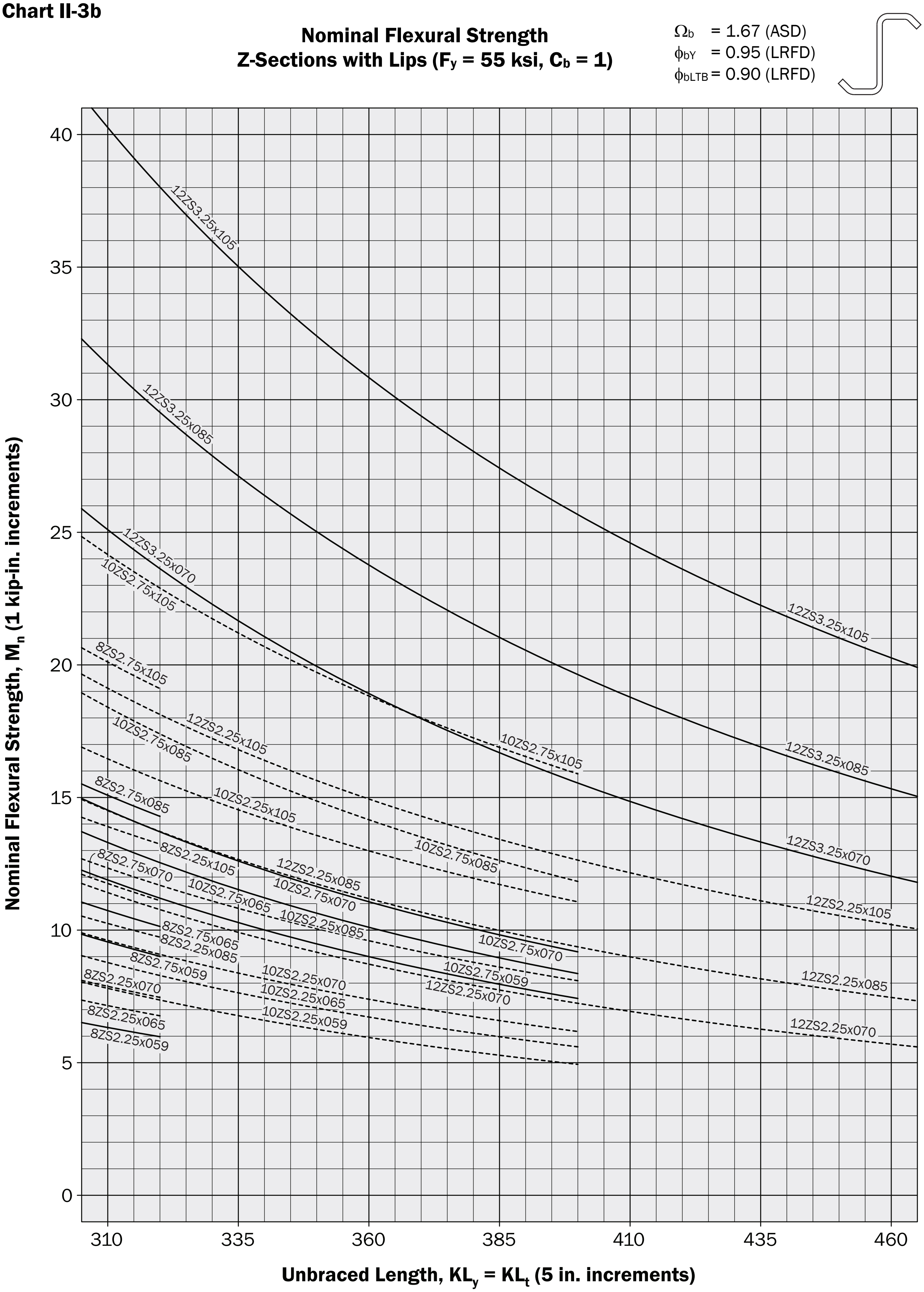


Chart II-3b

Nominal Flexural Strength
Z-Sections with Lips (F_y = 55 ksi, C_b = 1)

Ω_b = 1.67 (ASD)
ϕ_{bY} = 0.95 (LRFD)
ϕ_{bLTB} = 0.90 (LRFD)



SECTION 2 – COMBINED BENDING AND SHEAR

2.1 Notes on the Tables

(a) With the exception of the SSMA studs, the specific sections listed in these tables are not necessarily stock sections. They are included primarily as a guide in the design of cold-formed steel structural members.

(b) The section designations listed in these tables correspond to those for which dimensions and properties are given in Tables I-1, I-2 and I-4.

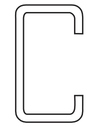
(c) Tables II-10a, II-11a and II-12a incorporate safety factors and are valid for ASD use only. Tables II-10b, II-11b and II-12b incorporate resistance factors and are valid for LRFD use only.

(d) The effects of standard factory punchouts in SSMA studs have been included in Tables II-11a, and II-11b. These punchouts are considered in SSMA studs with flange widths of 1.625 in. or less. Standard punchout sizes are 1.5 in. by 4.5 in. for sections with depths of 3.5 inches or more and 0.75 in. by 4.5 in. for sections with shallower depths. Punchouts are not included in the calculations for sections with web h/t ratios in excess of 200 due to the limits in Section B2.4. Those sections are marked with a "*".

2.2 Combined Shear and Bending Tables

Table II - 10a

ASD-Combined Shear and Bending[2,3,4]
C-Sections With Lips



Section	F_y = 33 ksi		F_y = 55 ksi		Section	F_y = 33 ksi		F_y = 55 ksi	
	V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.		V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.
12CS3.5x105	9.02	0.00	9.02	0.00	8CS3.5x105	9.43	0.00	12.2	0.00
	8.72	36.3	8.72	55.5		9.11	21.0	11.8	31.7
	7.82	70.2	7.82	107		8.16	40.6	10.5	61.3
	6.38	99.2	6.38	152		6.67	57.5	8.61	86.6
	4.51	122	4.51	186		4.71	70.4	6.08	106
	2.34	136	2.34	207		2.44	78.5	3.15	118
	0.00	140	0.00	215		0.00	81.3	0.00	123
12CS3.5x085	4.77	0.00	4.77	0.00	8CS3.5x085	6.18	0.00	7.33	0.00
	4.61	28.3	4.61	40.4		5.97	16.3	7.08	24.4
	4.13	54.7	4.13	78.1		5.35	31.5	6.35	47.1
	3.37	77.3	3.37	110		4.37	44.6	5.18	66.6
	2.39	94.7	2.39	135		3.09	54.6	3.67	81.5
	1.23	106	1.23	151		1.60	60.9	1.90	90.9
	0.00	109	0.00	156		0.00	63.1	0.00	94.1
12CS3.5x070	2.66	0.00	2.66	0.00	8CS3.5x070	4.08	0.00	4.08	0.00
	2.57	21.9	2.57	29.1		3.94	12.7	3.94	19.3
	2.30	42.4	2.30	56.3		3.53	24.6	3.53	37.3
	1.88	59.9	1.88	79.6		2.88	34.8	2.88	52.7
	1.33	73.4	1.33	97.5		2.04	42.7	2.04	64.5
	0.688	81.8	0.688	109		1.06	47.6	1.06	72.0
	0.00	84.7	0.00	113		0.00	49.3	0.00	74.5
10CS3.5x105	9.43	0.00	10.9	0.00	8CS3.5x065	3.26	0.00	3.26	0.00
	9.11	28.3	10.6	43.0		3.15	11.7	3.15	17.4
	8.16	54.7	9.48	83.1		2.82	22.6	2.82	33.6
	6.67	77.4	7.74	117		2.31	31.9	2.31	47.5
	4.71	94.7	5.47	144		1.63	39.1	1.63	58.1
	2.44	106	2.83	160		0.844	43.6	0.844	64.8
	0.00	109	0.00	166		0.00	45.1	0.00	67.1
10CS3.5x085	5.78	0.00	5.78	0.00	8CS3.5x059	2.43	0.00	2.43	0.00
	5.58	22.0	5.58	33.2		2.35	10.4	2.35	14.8
	5.01	42.5	5.01	64.1		2.11	20.1	2.11	28.6
	4.09	60.2	4.09	90.6		1.72	28.4	1.72	40.4
	2.89	73.7	2.89	111		1.22	34.8	1.22	49.5
	1.50	82.2	1.50	124		0.630	38.9	0.630	55.2
	0.00	85.1	0.00	128		0.00	40.2	0.00	57.2
10CS3.5x070	3.22	0.00	3.22	0.00	8CS2.5x105	9.43	0.00	12.2	0.00
	3.11	17.3	3.11	24.4		9.11	17.4	11.8	29.1
	2.79	33.4	2.79	47.2		8.16	33.7	10.5	56.2
	2.28	47.2	2.28	66.7		6.67	47.7	8.61	79.4
	1.61	57.8	1.61	81.7		4.71	58.4	6.08	97.3
	0.833	64.4	0.833	91.1		2.44	65.1	3.15	109
	0.00	66.7	0.00	94.3		0.00	67.4	0.00	112
10CS3.5x065	2.57	0.00	2.57	0.00	8CS2.5x085	6.18	0.00	7.33	0.00
	2.49	15.8	2.49	21.6		5.97	14.2	7.08	23.1
	2.23	30.6	2.23	41.7		5.35	27.5	6.35	44.6
	1.82	43.3	1.82	59.0		4.37	38.9	5.18	63.1
	1.29	53.0	1.29	72.2		3.09	47.6	3.67	77.3
	0.666	59.1	0.666	80.6		1.60	53.1	1.90	86.2
	0.00	61.2	0.00	83.4		0.00	55.0	0.00	89.3

Table II - 10a

ASD-Combined Shear and Bending[2,3,4]
C-Sections With Lips

Section	F_y = 33 ksi		F_y = 55 ksi		Section	F_y = 33 ksi		F_y = 55 ksi	
	V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.		V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.
8CS2.5x070	4.08	0.00	4.08	0.00	6CS2.5x059	2.98	0.00	3.32	0.00
	3.94	11.8	3.94	17.9		2.87	6.69	3.21	9.83
	3.53	22.7	3.53	34.5		2.58	12.9	2.87	19.0
	2.88	32.2	2.88	48.8		2.10	18.3	2.35	26.8
	2.04	39.4	2.04	59.8		1.49	22.4	1.66	32.9
	1.06	43.9	1.06	66.7		0.770	25.0	0.859	36.7
	0.00	45.5	0.00	69.0		0.00	25.8	0.00	38.0
8CS2.5x065	3.26	0.00	3.26	0.00	4CS2.5x105	4.44	0.00	7.40	0.00
	3.15	11.0	3.15	16.3		4.29	6.84	7.14	11.4
	2.82	21.2	2.82	31.5		3.84	13.2	6.40	22.0
	2.31	29.9	2.31	44.5		3.14	18.7	5.23	31.1
	1.63	36.6	1.63	54.5		2.22	22.9	3.70	38.1
	0.844	40.9	0.844	60.8		1.15	25.5	1.91	42.5
	0.00	42.3	0.00	63.0		0.00	26.4	0.00	44.0
8CS2.5x059	2.43	0.00	2.43	0.00	4CS2.5x085	3.63	0.00	6.06	0.00
	2.35	9.84	2.35	14.4		3.51	5.63	5.85	9.17
	2.11	19.0	2.11	27.8		3.15	10.9	5.25	17.7
	1.72	26.9	1.72	39.4		2.57	15.4	4.28	25.0
	1.22	32.9	1.22	48.2		1.82	18.8	3.03	30.7
	0.630	36.7	0.630	53.8		0.941	21.0	1.57	34.2
	0.00	38.0	0.00	55.7		0.00	21.7	0.00	35.4
6CS2.5x105	7.04	0.00	11.7	0.00	4CS2.5x070	3.02	0.00	5.03	0.00
	6.80	11.8	11.3	19.6		2.92	4.69	4.86	7.03
	6.09	22.7	10.2	37.9		2.61	9.06	4.36	13.6
	4.98	32.2	8.29	53.6		2.13	12.8	3.56	19.2
	3.52	39.4	5.86	65.7		1.51	15.7	2.52	23.5
	1.82	43.9	3.04	73.2		0.781	17.5	1.30	26.3
	0.00	45.5	0.00	75.8		0.00	18.1	0.00	27.2
6CS2.5x085	5.74	0.00	7.97	0.00	4CS2.5x065	2.81	0.00	4.66	0.00
	5.54	9.63	7.70	15.6		2.72	4.37	4.50	6.39
	4.97	18.6	6.91	30.2		2.43	8.45	4.04	12.4
	4.06	26.3	5.64	42.7		1.99	11.9	3.30	17.5
	2.87	32.2	3.99	52.3		1.41	14.6	2.33	21.4
	1.49	35.9	2.06	58.4		0.728	16.3	1.21	23.9
	0.00	37.2	0.00	60.4		0.00	16.9	0.00	24.7
6CS2.5x070	4.19	0.00	5.41	0.00	4CS2.5x059	2.56	0.00	3.84	0.00
	4.05	7.99	5.22	12.0		2.47	3.94	3.71	5.72
	3.63	15.4	4.68	23.3		2.22	7.62	3.33	11.1
	2.96	21.8	3.82	32.9		1.81	10.8	2.72	15.6
	2.09	26.7	2.70	40.3		1.28	13.2	1.92	19.1
	1.08	29.8	1.40	44.9		0.663	14.7	0.995	21.3
	0.00	30.9	0.00	46.5		0.00	15.2	0.00	22.1
6CS2.5x065	3.61	0.00	4.45	0.00					
	3.49	7.44	4.30	11.0					
	3.13	14.4	3.85	21.2					
	2.55	20.3	3.14	29.9					
	1.81	24.9	2.22	36.7					
	0.935	27.8	1.15	40.9					
	0.00	28.7	0.00	42.4					

Notes:

1. Shear and moment strengths have been divided by the appropriate safety factors. This table is for ASD use only.
2. Shear strengths have been calculated assuming no transverse stiffeners.
3. Linear interpolation between values is permitted.
4. The distortional buckling limit state is not considered in this table. Consideration of distortional buckling may result in lower strengths when restraint against distortional buckling is not provided. See Table II-7 for distortional buckling strengths.

Table II - 10b

LRFD-Combined Shear and Bending[2,3,4]
C-Sections With Lips

Section	F_y = 33 ksi		F_y = 55 ksi		Section	F_y = 33 ksi		F_y = 55 ksi	
	V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.		V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.
12CS3.5x105	13.7	0.00	13.7	0.00	8CS3.5x105	14.3	0.00	18.5	0.00
	13.2	57.6	13.2	88.1		13.8	33.4	17.9	50.3
	11.9	111	11.9	170		12.4	64.5	16.0	97.2
	9.70	157	9.70	241		10.1	91.2	13.1	137
	6.86	193	6.86	295		7.16	112	9.25	168
	3.55	215	3.55	329		3.71	125	4.79	188
	0.00	223	0.00	340		0.00	129	0.00	194
12CS3.5x085	7.25	0.00	7.25	0.00	8CS3.5x085	9.39	0.00	11.1	0.00
	7.00	44.9	7.00	64.1		9.07	25.9	10.8	38.6
	6.28	86.7	6.28	124		8.13	50.0	9.65	74.7
	5.13	123	5.13	175		6.64	70.8	7.88	106
	3.63	150	3.63	215		4.69	86.7	5.57	129
	1.88	168	1.88	239		2.43	96.7	2.88	144
	0.00	173	0.00	248		0.00	100	0.00	149
12CS3.5x070	4.04	0.00	4.04	0.00	8CS3.5x070	6.20	0.00	6.20	0.00
	3.90	34.8	3.90	46.2		5.99	20.2	5.99	30.6
	3.50	67.2	3.50	89.3		5.37	39.1	5.37	59.1
	2.86	95.1	2.86	126		4.38	55.3	4.38	83.6
	2.02	116	2.02	155		3.10	67.7	3.10	102
	1.05	130	1.05	172		1.60	75.5	1.60	114
	0.00	134	0.00	179		0.00	78.1	0.00	118
10CS3.5x105	14.3	0.00	16.6	0.00	8CS3.5x065	4.96	0.00	4.96	0.00
	13.8	44.9	16.1	68.2		4.79	18.5	4.79	27.6
	12.4	86.8	14.4	132		4.29	35.8	4.29	53.3
	10.1	123	11.8	186		3.50	50.6	3.50	75.3
	7.16	150	8.32	228		2.48	62.0	2.48	92.2
	3.71	168	4.30	255		1.28	69.1	1.28	103
	0.00	174	0.00	264		0.00	71.6	0.00	107
10CS3.5x085	8.79	0.00	8.79	0.00	8CS3.5x059	3.70	0.00	3.70	0.00
	8.49	34.9	8.49	52.6		3.57	16.5	3.57	23.5
	7.61	67.5	7.61	102		3.20	31.9	3.20	45.3
	6.21	95.4	6.21	144		2.62	45.1	2.62	64.1
	4.39	117	4.39	176		1.85	55.3	1.85	78.5
	2.27	130	2.27	196		0.958	61.6	0.958	87.6
	0.00	135	0.00	203		0.00	63.8	0.00	90.7
10CS3.5x070	4.89	0.00	4.89	0.00	8CS2.5x105	14.3	0.00	18.5	0.00
	4.72	27.4	4.72	38.7		13.8	27.7	17.9	46.1
	4.24	52.9	4.24	74.8		12.4	53.5	16.0	89.1
	3.46	74.8	3.46	106		10.1	75.6	13.1	126
	2.45	91.7	2.45	130		7.16	92.6	9.25	154
	1.27	102	1.27	145		3.71	103	4.79	172
	0.00	106	0.00	150		0.00	107	0.00	178
10CS3.5x065	3.91	0.00	3.91	0.00	8CS2.5x085	9.39	0.00	11.1	0.00
	3.78	25.1	3.78	34.2		9.07	22.6	10.8	36.7
	3.39	48.5	3.39	66.2		8.13	43.6	9.65	70.8
	2.77	68.6	2.77	93.6		6.64	61.6	7.88	100
	1.96	84.0	1.96	115		4.69	75.5	5.57	123
	1.01	93.7	1.01	128		2.43	84.2	2.88	137
	0.00	97.0	0.00	132		0.00	87.2	0.00	142

Table II - 10b

LRFD-Combined Shear and Bending[2,3,4]
C-Sections With Lips

Section	F_y = 33 ksi		F_y = 55 ksi		Section	F_y = 33 ksi		F_y = 55 ksi	
	V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.		V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.
8CS2.5x070	6.20	0.00	6.20	0.00	6CS2.5x059	4.52	0.00	5.04	0.00
	5.99	18.7	5.99	28.3		4.37	10.6	4.87	15.6
	5.37	36.1	5.37	54.8		3.92	20.5	4.37	30.1
	4.38	51.0	4.38	77.4		3.20	29.0	3.57	42.6
	3.10	62.5	3.10	94.8		2.26	35.5	2.52	52.2
	1.60	69.7	1.60	106		1.17	39.6	1.31	58.2
	0.00	72.2	0.00	110		0.00	41.0	0.00	60.2
8CS2.5x065	4.96	0.00	4.96	0.00	4CS2.5x105	6.74	0.00	11.2	0.00
	4.79	17.4	4.79	25.9		6.52	10.8	10.9	18.1
	4.29	33.6	4.29	50.0		5.84	21.0	9.74	34.9
	3.50	47.5	3.50	70.7		4.77	29.6	7.95	49.4
	2.48	58.1	2.48	86.5		3.37	36.3	5.62	60.5
	1.28	64.8	1.28	96.5		1.75	40.5	2.91	67.5
	0.00	67.1	0.00	99.9		0.00	41.9	0.00	69.9
8CS2.5x059	3.70	0.00	3.70	0.00	4CS2.5x085	5.52	0.00	9.21	0.00
	3.57	15.6	3.57	22.9		5.34	8.93	8.89	14.5
	3.20	30.2	3.20	44.2		4.78	17.2	7.97	28.1
	2.62	42.7	2.62	62.5		3.91	24.4	6.51	39.7
	1.85	52.2	1.85	76.5		2.76	29.9	4.60	48.7
	0.958	58.3	0.958	85.3		1.43	33.3	2.38	54.3
	0.00	60.3	0.00	88.4		0.00	34.5	0.00	56.2
6CS2.5x105	10.7	0.00	17.8	0.00	4CS2.5x070	4.59	0.00	7.65	0.00
	10.3	18.7	17.2	31.1		4.43	7.44	7.39	11.2
	9.26	36.1	15.4	60.1		3.97	14.4	6.62	21.6
	7.56	51.0	12.6	85.1		3.24	20.3	5.41	30.5
	5.35	62.5	8.91	104		2.29	24.9	3.82	37.3
	2.77	69.7	4.61	116		1.19	27.8	1.98	41.7
	0.00	72.2	0.00	120		0.00	28.7	0.00	43.1
6CS2.5x085	8.72	0.00	12.1	0.00	4CS2.5x065	4.27	0.00	7.09	0.00
	8.42	15.3	11.7	24.8		4.13	6.94	6.85	10.1
	7.55	29.5	10.5	47.9		3.70	13.4	6.14	19.6
	6.17	41.7	8.57	67.8		3.02	19.0	5.01	27.7
	4.36	51.1	6.06	83.0		2.14	23.2	3.54	33.9
	2.26	57.0	3.14	92.6		1.11	25.9	1.83	37.9
	0.00	59.0	0.00	95.9		0.00	26.8	0.00	39.2
6CS2.5x070	6.37	0.00	8.22	0.00	4CS2.5x059	3.89	0.00	5.84	0.00
	6.15	12.7	7.94	19.1		3.76	6.26	5.64	9.08
	5.51	24.5	7.12	36.9		3.37	12.1	5.06	17.5
	4.50	34.6	5.81	52.2		2.75	17.1	4.13	24.8
	3.18	42.4	4.11	63.9		1.95	20.9	2.92	30.4
	1.65	47.3	2.13	71.3		1.01	23.4	1.51	33.9
	0.00	49.0	0.00	73.8		0.00	24.2	0.00	35.1
6CS2.5x065	5.49	0.00	6.76	0.00					
	5.30	11.8	6.53	17.4					
	4.76	22.8	5.85	33.6					
	3.88	32.2	4.78	47.5					
	2.75	39.5	3.38	58.2					
	1.42	44.0	1.75	64.9					
	0.00	45.6	0.00	67.2					

Notes:

1. Shear and moment strengths have been multiplied by the appropriate resistance factors. This table is for LRFD use only.
2. Shear strengths have been calculated assuming no transverse stiffeners.
3. Linear interpolation between values is permitted.
4. The distortional buckling limit state is not considered in this table. Consideration of distortional buckling may result in lower strengths when restraint against distortional buckling is not provided. See Table II-7 for distortional buckling strengths.

Table II - 11a

ASD-Combined Shear and Bending[2,3,4]
SSMA Studs
C-Sections With Lips

Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi		Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi	
	V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.		V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.
1200S200-97	8.15	0.00	8.15	0.00	1000S200-97	8.84	0.00	9.86	0.00
	7.87	25.1	7.87	36.1		8.54	19.8	9.53	29.0
	7.05	48.4	7.05	69.8		7.66	38.2	8.54	56.0
	5.76	68.5	5.76	98.7		6.25	54.0	6.97	79.2
	4.07	83.8	4.07	121		4.42	66.2	4.93	97.0
	2.11	93.5	2.11	135		2.29	73.8	2.55	108
	0.00	96.8	0.00	140		0.00	76.4	0.00	112
1200S200-68	2.77	0.00	2.77	0.00	1000S200-68	3.35	0.00	3.35	0.00
	2.68	16.4	2.68	23.0		3.23	13.3	3.23	18.8
	2.40	31.8	2.40	44.4		2.90	25.8	2.90	36.2
	1.96	44.9	1.96	62.7		2.37	36.4	2.37	51.2
	1.38	55.0	1.38	76.8		1.67	44.6	1.67	62.8
	0.717	61.4	0.717	85.7		0.866	49.8	0.866	70.0
	0.00	63.5	0.00	88.7		0.00	51.5	0.00	72.5
1200S200-54*	1.38	0.00	1.38	0.00	1000S200-54	1.66	0.00	1.66	0.00
	1.33	12.4	1.33	16.1		1.60	10.1	1.60	13.2
	1.19	24.0	1.19	31.0		1.44	19.6	1.44	25.5
	0.974	33.9	0.974	43.9		1.17	27.7	1.17	36.1
	0.688	41.5	0.688	53.8		0.830	34.0	0.830	44.2
	0.356	46.3	0.356	60.0		0.430	37.9	0.430	49.3
	0.00	47.9	0.00	62.1		0.00	39.2	0.00	51.1
1000S250-97	8.84	0.00	9.86	0.00	1000S200-43*	0.836	0.00	-	-
	8.54	25.5	9.53	36.4		0.807	7.52	-	-
	7.66	49.2	8.54	70.3		0.724	14.5	-	-
	6.25	69.6	6.97	99.4		0.591	20.5	-	-
	4.42	85.2	4.93	122		0.418	25.2	-	-
	2.29	95.1	2.55	136		0.216	28.1	-	-
	0.00	98.4	0.00	141		0.00	29.1	-	-
1000S250-68	3.35	0.00	3.35	0.00	800S200-97	8.84	0.00	10.9	0.00
	3.23	17.1	3.23	21.5		8.54	16.9	10.5	25.0
	2.90	33.0	2.90	41.4		7.66	32.6	9.43	48.3
	2.37	46.6	2.37	58.6		6.25	46.1	7.70	68.3
	1.67	57.1	1.67	71.8		4.42	56.4	5.44	83.7
	0.866	63.7	0.866	80.1		2.29	62.9	2.82	93.3
	0.00	65.9	0.00	82.9		0.00	65.1	0.00	96.6
1000S250-54	1.66	0.00	1.66	0.00	800S200-68	4.22	0.00	4.22	0.00
	1.60	11.6	1.60	14.6		4.08	11.7	4.08	16.9
	1.44	22.5	1.44	28.1		3.65	22.6	3.65	32.6
	1.17	31.8	1.17	39.8		2.98	32.0	2.98	46.1
	0.830	39.0	0.830	48.7		2.11	39.2	2.11	56.5
	0.430	43.5	0.430	54.4		1.09	43.7	1.09	63.0
	0.00	45.0	0.00	56.3		0.00	45.3	0.00	65.2
1000S250-43*	0.836	0.00	-	-	800S200-54	2.09	0.00	2.09	0.00
	0.807	8.27	-	-		2.02	9.25	2.02	11.6
	0.724	16.0	-	-		1.81	17.9	1.81	22.4
	0.591	22.6	-	-		1.48	25.3	1.48	31.7
	0.418	27.7	-	-		1.05	31.0	1.05	38.9
	0.216	30.9	-	-		0.541	34.5	0.541	43.3
	0.00	32.0	-	-		0.00	35.8	0.00	44.9

Table II - 11a

ASD-Combined Shear and Bending[2,3,4]
SSMA Studs
C-Sections With Lips

Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi		Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi	
	V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.		V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.
800S200-43	1.05	0.00	-	-	600S200-97	6.91	0.00	10.5	0.00
	1.01	6.61	-	-		6.68	11.3	10.1	16.7
	0.910	12.8	-	-		5.99	21.7	9.07	32.3
	0.743	18.1	-	-		4.89	30.8	7.40	45.6
	0.525	22.1	-	-		3.46	37.7	5.24	55.9
	0.272	24.7	-	-		1.79	42.0	2.71	62.3
	0.00	25.5	-	-		0.00	43.5	0.00	64.5
800S200-33*	0.474	0.00	-	-	600S200-68	4.35	0.00	5.35	0.00
	0.458	4.17	-	-		4.20	7.87	5.17	11.3
	0.410	8.06	-	-		3.76	15.2	4.63	21.9
	0.335	11.4	-	-		3.07	21.5	3.78	30.9
	0.237	14.0	-	-		2.17	26.3	2.67	37.9
	0.123	15.6	-	-		1.13	29.4	1.38	42.2
	0.00	16.1	-	-		0.00	30.4	0.00	43.7
800S162-97	4.82	0.00	5.94	0.00	600S200-54	2.74	0.00	2.82	0.00
	4.66	15.1	5.74	18.8		2.65	6.23	2.73	7.87
	4.18	29.1	5.14	36.4		2.37	12.0	2.44	15.2
	3.41	41.2	4.20	51.4		1.94	17.0	2.00	21.5
	2.41	50.5	2.97	63.0		1.37	20.8	1.41	26.3
	1.25	56.3	1.54	70.2		0.709	23.3	0.731	29.4
	0.00	58.3	0.00	72.7		0.00	24.1	0.00	30.4
800S162-68	3.37	0.00	3.37	0.00	600S200-43	1.42	0.00	-	-
	3.25	8.88	3.25	12.9		1.37	4.46	-	-
	2.92	17.2	2.92	24.9		1.23	8.62	-	-
	2.38	24.3	2.38	35.2		1.00	12.2	-	-
	1.68	29.7	1.68	43.1		0.708	14.9	-	-
	0.871	33.2	0.871	48.1		0.366	16.7	-	-
	0.00	34.3	0.00	49.8		0.00	17.2	-	-
800S162-54	2.09	0.00	2.09	0.00	600S200-33	0.638	0.00	-	-
	2.02	6.82	2.02	9.52		0.616	3.18	-	-
	1.81	13.2	1.81	18.4		0.553	6.14	-	-
	1.48	18.6	1.48	26.0		0.451	8.68	-	-
	1.05	22.8	1.05	31.9		0.319	10.6	-	-
	0.541	25.5	0.541	35.5		0.165	11.9	-	-
	0.00	26.4	0.00	36.8		0.00	12.3	-	-
800S162-43	1.05	0.00	-	-	600S162-97	2.51	0.00	3.80	0.00
	1.01	5.21	-	-		2.43	9.93	3.68	14.7
	0.910	10.1	-	-		2.17	19.2	3.30	28.4
	0.743	14.2	-	-		1.78	27.1	2.69	40.1
	0.525	17.4	-	-		1.26	33.2	1.90	49.1
	0.272	19.5	-	-		0.650	37.1	0.985	54.8
	0.00	20.1	-	-		0.00	38.4	0.00	56.7
800S162-33*	0.474	0.00	-	-	600S162-68	2.34	0.00	2.88	0.00
	0.458	3.63	-	-		2.26	6.93	2.78	10.2
	0.410	7.01	-	-		2.03	13.4	2.49	19.7
	0.335	9.92	-	-		1.65	18.9	2.04	27.9
	0.237	12.1	-	-		1.17	23.2	1.44	34.2
	0.123	13.5	-	-		0.605	25.9	0.745	38.1
	0.00	14.0	-	-		0.00	26.8	0.00	39.5

Table II - 11a

ASD-Combined Shear and Bending[2,3,4]
SSMA Studs
C-Sections With Lips

Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi		Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi	
	V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.		V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.
600S162-54	1.89	0.00	1.95	0.00	400S162-68	0.895	0.00	1.36	0.00
	1.82	5.48	1.88	7.85		0.865	3.37	1.31	5.02
	1.64	10.6	1.69	15.2		0.775	6.50	1.17	9.70
	1.34	15.0	1.38	21.4		0.633	9.19	0.959	13.7
	0.945	18.3	0.973	26.3		0.447	11.3	0.678	16.8
	0.489	20.4	0.504	29.3		0.232	12.6	0.351	18.7
	0.00	21.2	0.00	30.3		0.00	13.0	0.00	19.4
600S162-43	1.24	0.00	-	-	400S162-54	0.944	0.00	1.22	0.00
	1.20	4.32	-	-		0.912	2.69	1.18	3.86
	1.07	8.34	-	-		0.818	5.19	1.06	7.45
	0.877	11.8	-	-		0.668	7.34	0.864	10.5
	0.620	14.4	-	-		0.472	9.00	0.611	12.9
	0.321	16.1	-	-		0.244	10.0	0.316	14.4
	0.00	16.7	-	-		0.00	10.4	0.00	14.9
600S162-33	0.638	0.00	-	-	400S162-43	0.809	0.00	-	-
	0.616	2.95	-	-		0.782	2.13	-	-
	0.553	5.70	-	-		0.701	4.12	-	-
	0.451	8.07	-	-		0.572	5.82	-	-
	0.319	9.88	-	-		0.405	7.13	-	-
	0.165	11.0	-	-		0.209	7.95	-	-
	0.00	11.4	-	-		0.00	8.23	-	-
550S162-68	2.06	0.00	2.53	0.00	400S162-33	0.595	0.00	-	-
	1.99	6.14	2.44	9.04		0.575	1.53	-	-
	1.78	11.9	2.19	17.5		0.515	2.95	-	-
	1.45	16.8	1.79	24.7		0.421	4.18	-	-
	1.03	20.5	1.27	30.3		0.298	5.12	-	-
	0.532	22.9	0.655	33.8		0.154	5.71	-	-
	0.00	23.7	0.00	34.9		0.00	5.91	-	-
550S162-54	1.67	0.00	1.88	0.00	362S162-68	0.663	0.00	1.00	0.00
	1.61	4.86	1.82	6.95		0.640	2.96	0.970	4.45
	1.44	9.38	1.63	13.4		0.574	5.72	0.869	8.59
	1.18	13.3	1.33	19.0		0.468	8.08	0.710	12.2
	0.833	16.2	0.940	23.3		0.331	9.90	0.502	14.9
	0.431	18.1	0.487	25.9		0.171	11.0	0.260	16.6
	0.00	18.8	0.00	26.9		0.00	11.4	0.00	17.2
550S162-43	1.20	0.00	-	-	362S162-54	0.706	0.00	1.02	0.00
	1.16	3.83	-	-		0.682	2.39	0.982	3.44
	1.04	7.40	-	-		0.611	4.61	0.880	6.64
	0.848	10.5	-	-		0.499	6.52	0.719	9.39
	0.599	12.8	-	-		0.353	7.98	0.508	11.5
	0.310	14.3	-	-		0.183	8.90	0.263	12.8
	0.00	14.8	-	-		0.00	9.22	0.00	13.3
550S162-33	0.698	0.00	-	-	362S162-43	0.676	0.00	-	-
	0.674	2.62	-	-		0.653	1.90	-	-
	0.605	5.06	-	-		0.585	3.67	-	-
	0.494	7.15	-	-		0.478	5.19	-	-
	0.349	8.76	-	-		0.338	6.36	-	-
	0.181	9.77	-	-		0.175	7.09	-	-
	0.00	10.1	-	-		0.00	7.34	-	-

Table II - 11a

ASD-Combined Shear and Bending[2,3,4]
SSMA Studs
C-Sections With Lips

Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi		Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi	
	V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.		V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.
362S162-33	0.521	0.00	-	-	250S162-68	0.343	0.00	0.519	0.00
	0.503	1.37	-	-		0.331	2.13	0.501	3.13
	0.451	2.65	-	-		0.297	4.11	0.449	6.05
	0.369	3.74	-	-		0.242	5.81	0.367	8.56
	0.261	4.58	-	-		0.171	7.11	0.259	10.5
	0.135	5.11	-	-		0.09	7.93	0.134	11.7
	0.00	5.29	-	-		0.00	8.21	0.00	12.1
350S162-68	0.592	0.00	0.897	0.00	250S162-54	0.373	0.00	0.564	0.00
	0.572	2.82	0.866	4.26		0.360	1.70	0.545	2.44
	0.513	5.45	0.777	8.22		0.323	3.28	0.489	4.71
	0.419	7.70	0.634	11.6		0.263	4.64	0.399	6.66
	0.296	9.43	0.448	14.2		0.186	5.69	0.282	8.16
	0.153	10.5	0.232	15.9		0.10	6.34	0.146	9.10
	0.00	10.9	0.00	16.4		0.00	6.57	0.00	9.42
350S162-54	0.633	0.00	0.947	0.00	250S162-43	0.394	0.00	-	-
	0.612	2.29	0.915	3.30		0.381	1.35	-	-
	0.548	4.41	0.820	6.37		0.342	2.61	-	-
	0.448	6.24	0.670	9.01		0.279	3.69	-	-
	0.317	7.65	0.473	11.0		0.197	4.52	-	-
	0.164	8.53	0.245	12.3		0.102	5.05	-	-
	0.00	8.83	0.00	12.7		0.00	5.22	-	-
350S162-43	0.631	0.00	-	-	250S162-33	0.399	0.00	-	-
	0.610	1.82	-	-		0.385	0.920	-	-
	0.547	3.52	-	-		0.345	1.78	-	-
	0.446	4.98	-	-		0.282	2.51	-	-
	0.316	6.10	-	-		0.199	3.08	-	-
	0.163	6.81	-	-		0.103	3.43	-	-
	0.00	7.05	-	-		0.00	3.55	-	-
350S162-33	0.487	0.00	-	-					
	0.470	1.32	-	-					
	0.422	2.54	-	-					
	0.344	3.60	-	-					
	0.243	4.40	-	-					
	0.126	4.91	-	-					
	0.00	5.08	-	-					

Notes:

1. Shear and moment strengths have been divided by the appropriate safety factors. This table is for ASD use only.
2. Shear strengths have been calculated assuming no transverse stiffeners.
3. Linear interpolation between values is permitted.
4. The distortional buckling limit state is not considered in this table. Consideration of distortional buckling may result in lower strengths when restraint against distortional buckling is not provided. See Table II-8 for distortional buckling strengths.

* Web h/t > 200, therefore bearing stiffeners are required.

Table II - 11b

LRFD-Combined Shear and Bending[2,3,4]
SSMA Studs
C-Sections With Lips

Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi		Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi	
	V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.		V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.
1200S200-97	12.4	0.00	12.4	0.00	1000S200-97	13.4	0.00	15.0	0.00
	12.0	39.8	12.0	57.3		13.0	31.4	14.5	46.0
	10.7	76.8	10.7	111		11.6	60.6	13.0	88.8
	8.75	109	8.75	157		9.50	85.7	10.6	126
	6.19	133	6.19	192		6.72	105	7.50	154
	3.20	148	3.20	214		3.48	117	3.88	172
	0.00	154	0.00	221		0.00	121	0.00	178
1200S200-68	4.21	0.00	4.21	0.00	1000S200-68	5.08	0.00	5.08	0.00
	4.07	26.1	4.07	36.4		4.91	21.2	4.91	29.8
	3.65	50.4	3.65	70.4		4.40	40.9	4.40	57.5
	2.98	71.3	2.98	99.5		3.60	57.8	3.60	81.3
	2.11	87.3	2.11	122		2.54	70.8	2.54	99.6
	1.09	97.4	1.09	136		1.32	78.9	1.32	111
	0.00	101	0.00	141		0.00	81.7	0.00	115
1200S200-54*	2.09	0.00	2.09	0.00	1000S200-54	2.52	0.00	2.52	0.00
	2.02	19.7	2.02	25.5		2.44	16.1	2.44	21.0
	1.81	38.0	1.81	49.2		2.19	31.1	2.19	40.5
	1.48	53.8	1.48	69.6		1.78	44.0	1.78	57.3
	1.05	65.9	1.05	85.3		1.26	53.9	1.26	70.1
	0.542	73.4	0.542	95.1		0.653	60.1	0.653	78.2
	0.00	76.0	0.00	98.5		0.00	62.2	0.00	81.0
1000S250-97	13.4	0.00	15.0	0.00	1000S200-43*	1.27	0.00	-	-
	13.0	40.4	14.5	57.7		1.23	11.9	-	-
	11.6	78.1	13.0	112		1.10	23.0	-	-
	9.50	110	10.6	158		0.898	32.6	-	-
	6.72	135	7.50	193		0.635	39.9	-	-
	3.48	151	3.88	216		0.329	44.5	-	-
	0.00	156	0.00	223		0.00	46.1	-	-
1000S250-68	5.08	0.00	5.08	0.00	800S200-97	13.4	0.00	16.5	0.00
	4.91	27.1	4.91	34.0		13.0	26.7	16.0	39.7
	4.40	52.3	4.40	65.8		11.6	51.7	14.3	76.7
	3.60	74.0	3.60	93.0		9.50	73.1	11.7	108
	2.54	90.6	2.54	114		6.72	89.5	8.27	133
	1.32	101	1.32	127		3.48	99.8	4.28	148
	0.00	105	0.00	132		0.00	103	0.00	153
1000S250-54	2.52	0.00	2.52	0.00	800S200-68	6.41	0.00	6.41	0.00
	2.44	18.5	2.44	23.1		6.20	18.6	6.20	26.8
	2.19	35.7	2.19	44.6		5.56	35.9	5.56	51.7
	1.78	50.5	1.78	63.1		4.54	50.8	4.54	73.2
	1.26	61.8	1.26	77.3		3.21	62.2	3.21	89.6
	0.653	69.0	0.653	86.2		1.66	69.4	1.66	99.9
	0.00	71.4	0.00	89.3		0.00	71.9	0.00	103
1000S250-43*	1.27	0.00	-	-	800S200-54	3.18	0.00	3.18	0.00
	1.23	13.1	-	-		3.07	14.7	3.07	18.4
	1.10	25.3	-	-		2.75	28.4	2.75	35.6
	0.898	35.8	-	-		2.25	40.1	2.25	50.3
	0.635	43.9	-	-		1.59	49.1	1.59	61.7
	0.329	49.0	-	-		0.822	54.8	0.822	68.8
	0.00	50.7	-	-		0.00	56.7	0.00	71.2

Table II - 11b

LRFD-Combined Shear and Bending[2,3,4]
SSMA Studs
C-Sections With Lips

Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi		Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi	
	V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.		V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.
800S200-43	1.60	0.00	-	-	600S200-97	10.5	0.00	15.9	0.00
	1.54	10.5	-	-		10.1	17.9	15.4	26.5
	1.38	20.3	-	-		9.10	34.5	13.8	51.2
	1.13	28.7	-	-		7.43	48.8	11.3	72.4
	0.798	35.1	-	-		5.25	59.8	7.96	88.7
	0.413	39.1	-	-		2.72	66.7	4.12	98.9
	0.00	40.5	-	-		0.00	69.0	0.00	102
800S200-33*	0.720	0.00	-	-	600S200-68	6.61	0.00	8.13	0.00
	0.696	6.62	-	-		6.38	12.5	7.85	17.9
	0.624	12.8	-	-		5.72	24.1	7.04	34.7
	0.509	18.1	-	-		4.67	34.1	5.75	49.0
	0.360	22.2	-	-		3.30	41.8	4.07	60.1
	0.186	24.7	-	-		1.71	46.6	2.10	67.0
	0.00	25.6	-	-		0.00	48.3	0.00	69.3
800S162-97	7.33	0.00	9.02	0.00	600S200-54	4.16	0.00	4.29	0.00
	7.08	23.9	8.72	29.9		4.02	9.89	4.14	12.5
	6.35	46.2	7.82	57.7		3.61	19.1	3.72	24.1
	5.18	65.4	6.38	81.6		2.94	27.0	3.03	34.1
	3.67	80.1	4.51	99.9		2.08	33.1	2.15	41.8
	1.90	89.3	2.34	111		1.08	36.9	1.11	46.6
	0.00	92.5	0.00	115		0.00	38.2	0.00	48.2
800S162-68	5.12	0.00	5.12	0.00	600S200-43	2.15	0.00	-	-
	4.94	14.1	4.94	20.4		2.08	7.08	-	-
	4.43	27.2	4.43	39.5		1.86	13.7	-	-
	3.62	38.5	3.62	55.9		1.52	19.3	-	-
	2.56	47.2	2.56	68.4		1.08	23.7	-	-
	1.32	52.6	1.32	76.3		0.557	26.4	-	-
	0.00	54.5	0.00	79.0		0.00	27.4	-	-
800S162-54	3.18	0.00	3.18	0.00	600S200-33	0.970	0.00	-	-
	3.07	10.8	3.07	15.1		0.937	5.04	-	-
	2.75	20.9	2.75	29.2		0.840	9.74	-	-
	2.25	29.6	2.25	41.3		0.686	13.8	-	-
	1.59	36.2	1.59	50.5		0.485	16.9	-	-
	0.822	40.4	0.822	56.4		0.251	18.8	-	-
	0.00	41.8	0.00	58.4		0.00	19.5	-	-
800S162-43	1.60	0.00	-	-	600S162-97	3.82	0.00	5.78	0.00
	1.54	8.27	-	-		3.69	15.8	5.59	23.3
	1.38	16.0	-	-		3.31	30.4	5.01	45.0
	1.13	22.6	-	-		2.70	43.1	4.09	63.6
	0.798	27.7	-	-		1.91	52.7	2.89	77.9
	0.413	30.9	-	-		0.988	58.8	1.50	86.9
	0.00	32.0	-	-		0.00	60.9	0.00	90.0
800S162-33*	0.720	0.00	-	-	600S162-68	3.55	0.00	4.38	0.00
	0.696	5.76	-	-		3.43	11.0	4.23	16.2
	0.624	11.1	-	-		3.08	21.2	3.79	31.3
	0.509	15.7	-	-		2.51	30.0	3.09	44.3
	0.360	19.3	-	-		1.78	36.8	2.19	54.2
	0.186	21.5	-	-		0.920	41.0	1.13	60.5
	0.00	22.3	-	-		0.00	42.5	0.00	62.6

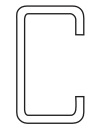
Table II - 11b

LRFD-Combined Shear and Bending[2,3,4]
SSMA Studs
C-Sections With Lips

Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi		Section	F_y = 33 ksi F_u = 45 ksi		F_y = 50 ksi F_u = 65 ksi	
	V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.		V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.
600S162-54	2.87	0.00	2.96	0.00	400S162-68	1.36	0.00	2.06	0.00
	2.77	8.69	2.86	12.5		1.31	5.34	1.99	7.97
	2.49	16.8	2.56	24.1		1.18	10.3	1.78	15.4
	2.03	23.7	2.09	34.0		0.962	14.6	1.46	21.8
	1.44	29.1	1.48	41.7		0.680	17.9	1.03	26.7
	0.743	32.4	0.766	46.5		0.352	19.9	0.533	29.7
	0.00	33.6	0.00	48.1		0.00	20.6	0.00	30.8
600S162-43	1.88	0.00	-	-	400S162-54	1.44	0.00	1.86	0.00
	1.82	6.85	-	-		1.39	4.27	1.79	6.12
	1.63	13.2	-	-		1.24	8.24	1.61	11.8
	1.33	18.7	-	-		1.02	11.7	1.31	16.7
	0.942	22.9	-	-		0.718	14.3	0.929	20.5
	0.488	25.6	-	-		0.372	15.9	0.481	22.8
	0.00	26.5	-	-		0.00	16.5	0.00	23.6
600S162-33	0.970	0.00	-	-	400S162-43	1.23	0.00	-	-
	0.937	4.68	-	-		1.19	3.38	-	-
	0.840	9.05	-	-		1.07	6.53	-	-
	0.686	12.8	-	-		0.870	9.24	-	-
	0.485	15.7	-	-		0.615	11.3	-	-
	0.251	17.5	-	-		0.318	12.6	-	-
	0.00	18.1	-	-		0.00	13.1	-	-
550S162-68	3.13	0.00	3.85	0.00	400S162-33	0.904	0.00	-	-
	3.02	9.74	3.72	14.3		0.874	2.43	-	-
	2.71	18.8	3.33	27.7		0.783	4.69	-	-
	2.21	26.6	2.72	39.2		0.640	6.63	-	-
	1.56	32.6	1.92	48.0		0.452	8.12	-	-
	0.809	36.4	0.996	53.6		0.234	9.05	-	-
	0.00	37.6	0.00	55.4		0.00	9.37	-	-
550S162-54	2.53	0.00	2.86	0.00	362S162-68	1.01	0.00	1.53	0.00
	2.45	7.70	2.76	11.0		0.973	4.69	1.47	7.06
	2.19	14.9	2.48	21.3		0.872	9.07	1.32	13.6
	1.79	21.0	2.02	30.1		0.712	12.8	1.08	19.3
	1.27	25.8	1.43	36.9		0.503	15.7	0.763	23.6
	0.655	28.8	0.740	41.2		0.261	17.5	0.395	26.3
	0.00	29.8	0.00	42.6		0.00	18.1	0.00	27.3
550S162-43	1.82	0.00	-	-	362S162-54	1.07	0.00	1.54	0.00
	1.76	6.08	-	-		1.04	3.79	1.49	5.45
	1.58	11.7	-	-		0.929	7.31	1.34	10.5
	1.29	16.6	-	-		0.758	10.3	1.09	14.9
	0.911	20.3	-	-		0.536	12.7	0.772	18.2
	0.472	22.7	-	-		0.278	14.1	0.400	20.4
	0.00	23.5	-	-		0.00	14.6	0.00	21.1
550S162-33	1.06	0.00	-	-	362S162-43	1.03	0.00	-	-
	1.02	4.15	-	-		0.992	3.02	-	-
	0.919	8.02	-	-		0.889	5.83	-	-
	0.750	11.3	-	-		0.726	8.24	-	-
	0.531	13.9	-	-		0.513	10.1	-	-
	0.275	15.5	-	-		0.266	11.3	-	-
	0.00	16.0	-	-		0.00	11.7	-	-

Table II - 11b

LRFD-Combined Shear and Bending[2,3,4]
SSMA Studs
C-Sections With Lips



Section	F_y = 33 ksi, F_u = 45 ksi		F_y = 50 ksi, F_u = 65 ksi		Section	F_y = 33 ksi, F_u = 45 ksi		F_y = 50 ksi, F_u = 65 ksi	
	V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.		V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.
362S162-33	0.792	0.00	-	-	250S162-68	0.521	0.00	0.789	0.00
	0.765	2.17	-	-		0.503	3.37	0.762	4.97
	0.686	4.20	-	-		0.451	6.51	0.683	9.60
	0.560	5.93	-	-		0.368	9.21	0.558	13.6
	0.396	7.27	-	-		0.260	11.3	0.394	16.6
	0.205	8.11	-	-		0.135	12.6	0.204	18.6
	0.00	8.39	-	-		0.00	13.0	0.00	19.2
350S162-68	0.900	0.00	1.36	0.00	250S162-54	0.566	0.00	0.858	0.00
	0.869	4.47	1.32	6.75		0.547	2.70	0.829	3.87
	0.779	8.64	1.18	13.0		0.490	5.21	0.743	7.47
	0.636	12.2	0.964	18.4		0.400	7.37	0.607	10.6
	0.450	15.0	0.682	22.6		0.283	9.02	0.429	12.9
	0.233	16.7	0.353	25.2		0.147	10.1	0.222	14.4
	0.00	17.3	0.00	26.1		0.00	10.4	0.00	14.9
350S162-54	0.962	0.00	1.44	0.00	250S162-43	0.599	0.00	-	-
	0.930	3.63	1.39	5.23		0.579	2.15	-	-
	0.833	7.00	1.25	10.1		0.519	4.14	-	-
	0.680	9.90	1.02	14.3		0.424	5.86	-	-
	0.481	12.1	0.720	17.5		0.300	7.18	-	-
	0.249	13.5	0.373	19.5		0.155	8.01	-	-
	0.00	14.0	0.00	20.2		0.00	8.29	-	-
350S162-43	0.960	0.00	-	-	250S162-33	0.606	0.00	-	-
	0.927	2.89	-	-		0.585	1.46	-	-
	0.831	5.59	-	-		0.525	2.82	-	-
	0.678	7.91	-	-		0.429	3.99	-	-
	0.480	9.68	-	-		0.303	4.88	-	-
	0.248	10.8	-	-		0.157	5.45	-	-
	0.00	11.2	-	-		0.00	5.64	-	-
350S162-33	0.740	0.00	-	-					
	0.715	2.09	-	-					
	0.641	4.03	-	-					
	0.523	5.70	-	-					
	0.370	6.99	-	-					
	0.192	7.79	-	-					
	0.00	8.07	-	-					

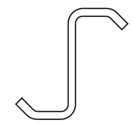
Notes:

1. Shear and moment strengths have been multiplied by the appropriate resistance factors. This table is for LRFD use only.
2. Shear strengths have been calculated assuming no transverse stiffeners.
3. Linear interpolation between values is permitted.
4. The distortional buckling limit state is not considered in this table. Consideration of distortional buckling may result in lower strengths when restraint against distortional buckling is not provided. See Table II-8 for distortional buckling strengths.

* Web h/t > 200, therefore bearing stiffeners are required.

Table II - 12a

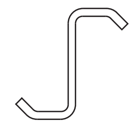
ASD-Combined Shear and Bending[2,3,4]
Z-Sections With Lips



Section	F_y = 33 ksi		F_y = 55 ksi		Section	F_y = 33 ksi		F_y = 55 ksi	
	V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.		V [1] kips	M [1] kip-in.	V [1] kips	M [1] kip-in.
12ZS3.25x105	9.02	0.00	9.02	0.00	10ZS2.75x085	5.78	0.00	5.78	0.00
	8.72	37.3	8.72	56.4		5.58	21.5	5.58	33.0
	7.82	72.0	7.82	109		5.01	41.6	5.01	63.7
	6.38	102	6.38	154		4.09	58.8	4.09	90.1
	4.51	125	4.51	189		2.89	72.0	2.89	110
	2.34	139	2.34	210		1.50	80.3	1.50	123
	0.00	144	0.00	218		0.00	83.1	0.00	127
12ZS3.25x085	4.77	0.00	4.77	0.00	10ZS2.75x070	3.22	0.00	3.22	0.00
	4.61	29.4	4.61	41.2		3.11	17.8	3.11	25.0
	4.13	56.8	4.13	79.7		2.79	34.3	2.79	48.3
	3.37	80.3	3.37	113		2.28	48.5	2.28	68.3
	2.39	98.4	2.39	138		1.61	59.5	1.61	83.7
	1.23	110	1.23	154		0.833	66.3	0.833	93.3
	0.00	114	0.00	159		0.00	68.7	0.00	96.6
12ZS3.25x070	2.66	0.00	2.66	0.00	10ZS2.75x065	2.57	0.00	2.57	0.00
	2.57	23.0	2.57	30.2		2.49	16.2	2.49	22.3
	2.30	44.4	2.30	58.3		2.23	31.3	2.23	43.0
	1.88	62.8	1.88	82.4		1.82	44.3	1.82	60.9
	1.33	76.9	1.33	101		1.29	54.3	1.29	74.6
	0.688	85.8	0.688	113		0.666	60.5	0.666	83.2
	0.00	88.8	0.00	117		0.00	62.7	0.00	86.1
12ZS2.25x105	9.02	0.00	9.02	0.00	10ZS2.75x059	1.92	0.00	1.92	0.00
	8.72	30.3	8.72	47.9		1.86	14.5	1.86	19.2
	7.82	58.5	7.82	92.6		1.66	27.9	1.66	37.2
	6.38	82.7	6.38	131		1.36	39.5	1.36	52.6
	4.51	101	4.51	160		0.961	48.4	0.961	64.4
	2.34	113	2.34	179		0.498	53.9	0.498	71.8
	0.00	117	0.00	185		0.00	55.9	0.00	74.3
12ZS2.25x085	4.77	0.00	4.77	0.00	10ZS2.25x105	9.43	0.00	10.9	0.00
	4.61	23.5	4.61	37.1		9.11	23.8	10.6	38.6
	4.13	45.5	4.13	71.6		8.16	46.0	9.48	74.5
	3.37	64.3	3.37	101		6.67	65.1	7.74	105
	2.39	78.8	2.39	124		4.71	79.7	5.47	129
	1.23	87.8	1.23	138		2.44	88.9	2.83	144
	0.00	90.9	0.00	143		0.00	92.0	0.00	149
12ZS2.25x070	2.66	0.00	2.66	0.00	10ZS2.25x085	5.78	0.00	5.78	0.00
	2.57	18.6	2.57	28.5		5.58	18.9	5.58	30.1
	2.30	35.9	2.30	55.0		5.01	36.6	5.01	58.2
	1.88	50.8	1.88	77.8		4.09	51.7	4.09	82.3
	1.33	62.2	1.33	95.3		2.89	63.4	2.89	101
	0.688	69.4	0.688	106		1.50	70.7	1.50	112
	0.00	71.8	0.00	110		0.00	73.2	0.00	116
10ZS2.75x105	9.43	0.00	10.9	0.00	10ZS2.25x070	3.22	0.00	3.22	0.00
	9.11	26.5	10.6	43.6		3.11	15.1	3.11	23.3
	8.16	51.1	9.48	84.3		2.79	29.1	2.79	45.0
	6.67	72.3	7.74	119		2.28	41.2	2.28	63.6
	4.71	88.5	5.47	146		1.61	50.5	1.61	78.0
	2.44	98.7	2.83	163		0.833	56.3	0.833	86.9
	0.00	102	0.00	169		0.00	58.3	0.00	90.0

Table II - 12a

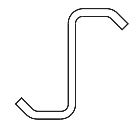
ASD-Combined Shear and Bending[2,3,4]
Z-Sections With Lips



Section	F_y = 33 ksi		F_y = 55 ksi		Section	F_y = 33 ksi		F_y = 55 ksi	
	V[1] kips	M[1] kip-in.	V[1] kips	M[1] kip-in.		V[1] kips	M[1] kip-in.	V[1] kips	M[1] kip-in.
10ZS2.25x065	2.57	0.00	2.57	0.00	8ZS2.25x105	9.43	0.00	12.2	0.00
	2.49	13.8	2.49	20.3		9.11	17.4	11.8	29.0
	2.23	26.7	2.23	39.2		8.16	33.7	10.5	56.1
	1.82	37.8	1.82	55.4		6.67	47.6	8.61	79.3
	1.29	46.2	1.29	67.9		4.71	58.3	6.08	97.2
	0.666	51.6	0.666	75.7		2.44	65.0	3.15	108
	0.00	53.4	0.00	78.4		0.00	67.3	0.00	112
10ZS2.25x059	1.92	0.00	1.92	0.00	8ZS2.25x085	6.18	0.00	7.33	0.00
	1.86	12.3	1.86	17.7		5.97	14.2	7.08	23.7
	1.66	23.8	1.66	34.2		5.35	27.4	6.35	45.7
	1.36	33.6	1.36	48.4		4.37	38.8	5.18	64.6
	0.961	41.2	0.961	59.3		3.09	47.5	3.67	79.1
	0.498	45.9	0.498	66.1		1.60	53.0	1.90	88.3
	0.00	47.5	0.00	68.5		0.00	54.8	0.00	91.4
8ZS2.75x105	9.43	0.00	12.2	0.00	8ZS2.25x070	4.08	0.00	4.08	0.00
	9.11	19.5	11.8	32.2		3.94	11.7	3.94	19.1
	8.16	37.7	10.5	62.2		3.53	22.7	3.53	37.0
	6.67	53.3	8.61	87.9		2.88	32.1	2.88	52.3
	4.71	65.3	6.08	108		2.04	39.3	2.04	64.1
	2.44	72.8	3.15	120		1.06	43.8	1.06	71.5
	0.00	75.4	0.00	124		0.00	45.4	0.00	74.0
8ZS2.75x085	6.18	0.00	7.33	0.00	8ZS2.25x065	3.26	0.00	3.26	0.00
	5.97	15.9	7.08	24.2		3.15	10.9	3.15	17.1
	5.35	30.7	6.35	46.8		2.82	21.1	2.82	33.0
	4.37	43.4	5.18	66.2		2.31	29.8	2.31	46.7
	3.09	53.2	3.67	81.1		1.63	36.5	1.63	57.2
	1.60	59.3	1.90	90.4		0.844	40.7	0.844	63.8
	0.00	61.4	0.00	93.6		0.00	42.2	0.00	66.1
8ZS2.75x070	4.08	0.00	4.08	0.00	8ZS2.25x059	2.43	0.00	2.43	0.00
	3.94	13.1	3.94	19.2		2.35	9.93	2.35	15.3
	3.53	25.4	3.53	37.0		2.11	19.2	2.11	29.6
	2.88	35.9	2.88	52.4		1.72	27.1	1.72	41.8
	2.04	44.0	2.04	64.2		1.22	33.2	1.22	51.2
	1.06	49.1	1.06	71.6		0.630	37.0	0.630	57.1
	0.00	50.8	0.00	74.1		0.00	38.3	0.00	59.1
8ZS2.75x065	3.26	0.00	3.26	0.00	6ZS2.25x105	7.04	0.00	11.7	0.00
	3.15	12.0	3.15	17.7		6.80	11.8	11.3	19.6
	2.82	23.1	2.82	34.2		6.09	22.7	10.2	37.8
	2.31	32.7	2.31	48.3		4.98	32.1	8.29	53.5
	1.63	40.1	1.63	59.2		3.52	39.3	5.86	65.5
	0.844	44.7	0.844	66.0		1.82	43.9	3.04	73.1
	0.00	46.3	0.00	68.3		0.00	45.4	0.00	75.7
8ZS2.75x059	2.43	0.00	2.43	0.00	6ZS2.25x085	5.74	0.00	7.97	0.00
	2.35	10.7	2.35	15.5		5.54	9.59	7.70	16.0
	2.11	20.6	2.11	30.0		4.97	18.5	6.91	30.9
	1.72	29.2	1.72	42.4		4.06	26.2	5.64	43.7
	1.22	35.7	1.22	52.0		2.87	32.1	3.99	53.5
	0.630	39.8	0.630	57.9		1.49	35.8	2.06	59.7
	0.00	41.2	0.00	60.0		0.00	37.1	0.00	61.8

Table II - 12a

ASD-Combined Shear and Bending[2,3,4]
Z-Sections With Lips



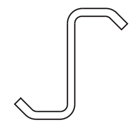
Section	F_y = 33 ksi		F_y = 55 ksi		Section	F_y = 33 ksi		F_y = 55 ksi	
	V[1] kips	M[1] kip-in.	V[1] kips	M[1] kip-in.		V[1] kips	M[1] kip-in.	V[1] kips	M[1] kip-in.
6ZS2.25x070	4.19	0.00	5.41	0.00	4ZS2.25x070	3.02	0.00	5.03	0.00
	4.05	7.95	5.22	13.0		2.92	4.65	4.86	7.60
	3.63	15.4	4.68	25.1		2.61	8.99	4.36	14.7
	2.96	21.7	3.82	35.4		2.13	12.7	3.56	20.8
	2.09	26.6	2.70	43.4		1.51	15.6	2.52	25.4
	1.08	29.7	1.40	48.4		0.781	17.4	1.30	28.4
	0.00	30.7	0.00	50.1		0.00	18.0	0.00	29.4
6ZS2.25x065	3.61	0.00	4.45	0.00	4ZS2.25x065	2.81	0.00	4.66	0.00
	3.49	7.40	4.30	11.6		2.72	4.34	4.50	6.76
	3.13	14.3	3.85	22.3		2.43	8.38	4.04	13.1
	2.55	20.2	3.14	31.6		1.99	11.9	3.30	18.5
	1.81	24.8	2.22	38.6		1.41	14.5	2.33	22.6
	0.935	27.6	1.15	43.1		0.728	16.2	1.21	25.2
	0.00	28.6	0.00	44.6		0.00	16.8	0.00	26.1
6ZS2.25x059	2.98	0.00	3.32	0.00	4ZS2.25x059	2.56	0.00	3.84	0.00
	2.87	6.73	3.21	10.3		2.47	3.95	3.71	6.04
	2.58	13.0	2.87	20.0		2.22	7.63	3.33	11.7
	2.10	18.4	2.35	28.2		1.81	10.8	2.72	16.5
	1.49	22.5	1.66	34.6		1.28	13.2	1.92	20.2
	0.770	25.1	0.859	38.5		0.663	14.7	0.995	22.5
	0.00	26.0	0.00	39.9		0.00	15.3	0.00	23.3

Notes:

1. Shear and moment strengths have been divided by the appropriate safety factors. This table is for ASD use only.
2. Shear strengths have been calculated assuming no transverse stiffeners.
3. Linear interpolation between values is permitted.
4. The distortional buckling limit state is not considered in this table. Consideration of distortional buckling may result in lower strengths when restraint against distortional buckling is not provided. See Table II-9 for distortional buckling strengths.

Table II - 12b

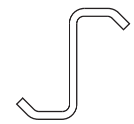
LRFD-Combined Shear and Bending[2,3,4]
Z-Sections With Lips



Section	F_y = 33 ksi		F_y = 55 ksi		Section	F_y = 33 ksi		F_y = 55 ksi	
	V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.		V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.
12ZS3.25x105	13.7	0.00	13.7	0.00	10ZS2.75x085	8.79	0.00	8.79	0.00
	13.2	59.1	13.2	89.4		8.49	34.1	8.49	52.3
	11.9	114	11.9	173		7.61	65.9	7.61	101
	9.70	162	9.70	244		6.21	93.2	6.21	143
	6.86	198	6.86	299		4.39	114	4.39	175
	3.55	221	3.55	334		2.27	127	2.27	195
	0.00	228	0.00	345		0.00	132	0.00	202
12ZS3.25x085	7.25	0.00	7.25	0.00	10ZS2.75x070	4.89	0.00	4.89	0.00
	7.00	46.6	7.00	65.4		4.72	28.2	4.72	39.7
	6.28	90.1	6.28	126		4.24	54.5	4.24	76.6
	5.13	127	5.13	179		3.46	77.0	3.46	108
	3.63	156	3.63	219		2.45	94.3	2.45	133
	1.88	174	1.88	244		1.27	105	1.27	148
	0.00	180	0.00	253		0.00	109	0.00	153
12ZS3.25x070	4.04	0.00	4.04	0.00	10ZS2.75x065	3.91	0.00	3.91	0.00
	3.90	36.5	3.90	47.9		3.78	25.7	3.78	35.3
	3.50	70.5	3.50	92.5		3.39	49.7	3.39	68.3
	2.86	99.6	2.86	131		2.77	70.3	2.77	96.6
	2.02	122	2.02	160		1.96	86.1	1.96	118
	1.05	136	1.05	179		1.01	96.0	1.01	132
	0.00	141	0.00	185		0.00	99.4	0.00	137
12ZS2.25x105	13.7	0.00	13.7	0.00	10ZS2.75x059	2.92	0.00	2.92	0.00
	13.2	48.0	13.2	76.0		2.82	22.9	2.82	30.5
	11.9	92.8	11.9	147		2.53	44.3	2.53	59.0
	9.70	131	9.70	208		2.07	62.7	2.07	83.4
	6.86	161	6.86	254		1.46	76.7	1.46	102
	3.55	179	3.55	284		0.756	85.6	0.756	114
	0.00	186	0.00	294		0.00	88.6	0.00	118
12ZS2.25x085	7.25	0.00	7.25	0.00	10ZS2.25x105	14.3	0.00	16.6	0.00
	7.00	37.3	7.00	58.8		13.8	37.8	16.1	61.2
	6.28	72.1	6.28	114		12.4	73.0	14.4	118
	5.13	102	5.13	161		10.1	103	11.8	167
	3.63	125	3.63	197		7.16	126	8.32	205
	1.88	139	1.88	220		3.71	141	4.30	228
	0.00	144	0.00	227		0.00	146	0.00	236
12ZS2.25x070	4.04	0.00	4.04	0.00	10ZS2.25x085	8.79	0.00	8.79	0.00
	3.90	29.5	3.90	45.2		8.49	30.0	8.49	47.8
	3.50	57.0	3.50	87.3		7.61	58.0	7.61	92.3
	2.86	80.6	2.86	123		6.21	82.1	6.21	131
	2.02	98.7	2.02	151		4.39	101	4.39	160
	1.05	110	1.05	169		2.27	112	2.27	178
	0.00	114	0.00	175		0.00	116	0.00	185
10ZS2.75x105	14.3	0.00	16.6	0.00	10ZS2.25x070	4.89	0.00	4.89	0.00
	13.8	42.0	16.1	69.2		4.72	23.9	4.72	37.0
	12.4	81.1	14.4	134		4.24	46.2	4.24	71.4
	10.1	115	11.8	189		3.46	65.4	3.46	101
	7.16	140	8.32	232		2.45	80.1	2.45	124
	3.71	157	4.30	258		1.27	89.3	1.27	138
	0.00	162	0.00	268		0.00	92.5	0.00	143

Table II - 12b

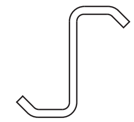
LRFD-Combined Shear and Bending[2,3,4]
Z-Sections With Lips



Section	F_y = 33 ksi		F_y = 55 ksi		Section	F_y = 33 ksi		F_y = 55 ksi	
	V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.		V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.
10ZS2.25x065	3.91	0.00	3.91	0.00	8ZS2.25x105	14.3	0.00	18.5	0.00
	3.78	21.9	3.78	32.2		13.8	27.6	17.9	46.1
	3.39	42.4	3.39	62.2		12.4	53.4	16.0	89.0
	2.77	59.9	2.77	88.0		10.1	75.5	13.1	126
	1.96	73.4	1.96	108		7.16	92.5	9.25	154
	1.01	81.8	1.01	120		3.71	103	4.79	172
	0.00	84.7	0.00	124		0.00	107	0.00	178
10ZS2.25x059	2.92	0.00	2.92	0.00	8ZS2.25x085	9.39	0.00	11.1	0.00
	2.82	19.5	2.82	28.1		9.07	22.5	10.8	37.5
	2.53	37.7	2.53	54.3		8.13	43.5	9.65	72.5
	2.07	53.3	2.07	76.8		6.64	61.5	7.88	103
	1.46	65.3	1.46	94.1		4.69	75.3	5.57	126
	0.756	72.9	0.756	105		2.43	84.0	2.88	140
	0.00	75.4	0.00	109		0.00	87.0	0.00	145
8ZS2.75x105	14.3	0.00	18.5	0.00	8ZS2.25x070	6.20	0.00	6.20	0.00
	13.8	31.0	17.9	51.1		5.99	18.6	5.99	30.4
	12.4	59.8	16.0	98.7		5.37	36.0	5.37	58.7
	10.1	84.6	13.1	140		4.38	50.9	4.38	83.0
	7.16	104	9.25	171		3.10	62.3	3.10	102
	3.71	116	4.79	191		1.60	69.5	1.60	113
	0.00	120	0.00	197		0.00	72.0	0.00	117
8ZS2.75x085	9.39	0.00	11.1	0.00	8ZS2.25x065	4.96	0.00	4.96	0.00
	9.07	25.2	10.8	38.4		4.79	17.3	4.79	27.1
	8.13	48.7	9.65	74.3		4.29	33.5	4.29	52.4
	6.64	68.9	7.88	105		3.50	47.3	3.50	74.1
	4.69	84.4	5.57	129		2.48	58.0	2.48	90.8
	2.43	94.1	2.88	143		1.28	64.6	1.28	101
	0.00	97.4	0.00	149		0.00	66.9	0.00	105
8ZS2.75x070	6.20	0.00	6.20	0.00	8ZS2.25x059	3.70	0.00	3.70	0.00
	5.99	20.9	5.99	30.4		3.57	15.7	3.57	24.3
	5.37	40.3	5.37	58.8		3.20	30.4	3.20	46.9
	4.38	57.0	4.38	83.1		2.62	43.0	2.62	66.3
	3.10	69.8	3.10	102		1.85	52.7	1.85	81.3
	1.60	77.8	1.60	114		0.958	58.8	0.958	90.6
	0.00	80.6	0.00	118		0.00	60.8	0.00	93.8
8ZS2.75x065	4.96	0.00	4.96	0.00	6ZS2.25x105	10.7	0.00	17.8	0.00
	4.79	19.0	4.79	28.0		10.3	18.6	17.2	31.1
	4.29	36.7	4.29	54.2		9.26	36.0	15.4	60.0
	3.50	51.9	3.50	76.6		7.56	50.9	12.6	84.9
	2.48	63.6	2.48	93.8		5.35	62.4	8.91	104
	1.28	70.9	1.28	105		2.77	69.6	4.61	116
	0.00	73.4	0.00	108		0.00	72.0	0.00	120
8ZS2.75x059	3.70	0.00	3.70	0.00	6ZS2.25x085	8.72	0.00	12.1	0.00
	3.57	16.9	3.57	24.6		8.42	15.2	11.7	25.4
	3.20	32.7	3.20	47.6		7.55	29.4	10.5	49.0
	2.62	46.3	2.62	67.3		6.17	41.6	8.57	69.3
	1.85	56.7	1.85	82.4		4.36	50.9	6.06	84.9
	0.958	63.2	0.958	91.9		2.26	56.8	3.14	94.7
	0.00	65.4	0.00	95.2		0.00	58.8	0.00	98.0

Table II - 12b

LRFD-Combined Shear and Bending[2,3,4]
Z-Sections With Lips



Section	F_y = 33 ksi		F_y = 55 ksi		Section	F_y = 33 ksi		F_y = 55 ksi	
	V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.		V_u [1] kips	M_u [1] kip-in.	V_u [1] kips	M_u [1] kip-in.
6ZS2.25x070	6.37	0.00	8.22	0.00	4ZS2.25x070	4.59	0.00	7.65	0.00
	6.15	12.6	7.94	20.6		4.43	7.38	7.39	12.1
	5.51	24.4	7.12	39.7		3.97	14.3	6.62	23.3
	4.50	34.5	5.81	56.2		3.24	20.2	5.41	32.9
	3.18	42.2	4.11	68.8		2.29	24.7	3.82	40.3
	1.65	47.1	2.13	76.8		1.19	27.6	1.98	45.0
	0.00	48.7	0.00	79.5		0.00	28.5	0.00	46.6
6ZS2.25x065	5.49	0.00	6.76	0.00	4ZS2.25x065	4.27	0.00	7.09	0.00
	5.30	11.7	6.53	18.3		4.13	6.88	6.85	10.7
	4.76	22.7	5.85	35.4		3.70	13.3	6.14	20.7
	3.88	32.1	4.78	50.1		3.02	18.8	5.01	29.3
	2.75	39.3	3.38	61.3		2.14	23.0	3.54	35.9
	1.42	43.8	1.75	68.4		1.11	25.7	1.83	40.0
	0.00	45.4	0.00	70.8		0.00	26.6	0.00	41.5
6ZS2.25x059	4.52	0.00	5.04	0.00	4ZS2.25x059	3.89	0.00	5.84	0.00
	4.37	10.7	4.87	16.4		3.76	6.27	5.64	9.58
	3.92	20.6	4.37	31.7		3.37	12.1	5.06	18.5
	3.20	29.2	3.57	44.8		2.75	17.1	4.13	26.2
	2.26	35.7	2.52	54.8		1.95	21.0	2.92	32.1
	1.17	39.9	1.31	61.2		1.01	23.4	1.51	35.8
	0.00	41.3	0.00	63.3		0.00	24.2	0.00	37.0

Notes:

1. Shear and moment strengths have been multiplied by the appropriate resistance factors. This table is for LRFD use only.
2. Shear strengths have been calculated assuming no transverse stiffeners.
3. Linear interpolation between values is permitted
4. The distortional buckling limit state is not considered in this table. Consideration of distortional buckling may result in lower strengths when restraint against distortional buckling is not provided. See Table II-9 for distortional buckling strengths.

SECTION 3 – WEB CRIPPLING

3.1 Notes on the Tables

(a) With the exception of the SSMA studs, the specific sections listed in these tables are not necessarily stock sections. They are included primarily as a guide in the design of cold-formed steel structural members.

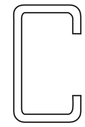
(b) The section designations listed in these tables correspond to those for which dimensions and properties are given in Tables I-1, I-2 and I-4.

(c) The values provided in Tables II-13, II-14 and II-15 are nominal strengths which do not incorporate safety or resistance factors. These nominal strengths must be modified by safety factors (ASD) or resistance factors (LRFD) for use in design. See the appropriate *Specification* sections for more information.

(d) The nominal crippling strengths of SSMA studs in Table II-14 do not include any allowance for the effects of standard punchouts. These punchouts are present in SSMA studs with flange widths of 1.625 in. or less. Standard punchout sizes are 1.5 in. by 4.5 in. for sections with depths of 3.5 inches or more and 0.75 in. by 4.5 in. for sections with shallower depths. Tables II-16a and II-16b provide modification factors that can be used to calculate the reduced crippling strength taking into account the standard punchouts in cases where the punchouts do not overlap the bearing length, N.

3.2 Web Crippling Tables

Table II - 13

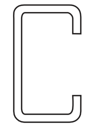


Web Crippling, P_n, kips [1,2,4]
C-Sections With Lips

Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi				F_y = 55 ksi				Ω_w	ϕ_w
			Bearing Length, N, in.[4]				Bearing Length, N, in.[4]					
			1	2	4	6	1	2	4	6		
12CS3.5x105	Fastened	A	1.95	2.37	2.96	3.41	3.25	3.94	4.93	5.69	1.75	0.85
		B	4.20	4.73	5.47	6.04	7.00	7.88	9.12	10.1	1.65	0.90
		C	1.67	1.85	2.12	2.32	2.78	3.09	3.53	3.87	1.75	0.85
		D	5.32	5.76	6.37	6.85	8.87	9.59	10.6	11.4	1.75	0.85
	Unfastened	A	1.95	2.37	2.96	3.41	3.25	3.94	4.93	5.69	1.85	0.80
		B	4.20	4.73	5.47	6.04	7.00	7.88	9.12	10.1	1.65	0.90
		C	1.82	1.92	2.07	2.17	3.04	3.20	3.44	3.62	1.65	0.90
		D	3.86	4.36	5.08	5.63	6.43	7.27	8.46	9.38	1.90	0.80
12CS3.5x085	Fastened	A	1.28	1.56	1.97	2.29	2.13	2.61	3.29	3.81	1.75	0.85
		B	2.67	3.03	3.54	3.93	4.45	5.05	5.89	6.54	1.65	0.90
		C	0.985	1.10	1.27	1.40	1.64	1.84	2.12	2.34	1.75	0.85
		D	3.31	3.61	4.03	4.35	5.52	6.01	6.71	7.24	1.75	0.85
	Unfastened	A	1.28	1.56	1.97	2.29	2.13	2.61	3.29	3.81	1.85	0.80
		B	2.67	3.03	3.54	3.93	4.45	5.05	5.89	6.54	1.65	0.90
		C	1.02	1.08	1.17	1.24	1.70	1.80	1.95	2.06	1.65	0.90
		D	1.95	2.22	2.61	2.91	3.25	3.71	4.35	4.85	1.90	0.80
12CS3.5x070	Fastened	A	0.861	1.06	1.35	1.57	1.44	1.77	2.25	2.62	1.75	0.85
		B	1.75	2.00	2.35	2.62	2.91	3.33	3.92	4.37	1.65	0.90
		C	0.590	0.666	0.774	0.857	0.983	1.11	1.29	1.43	1.75	0.85
		D	2.12	2.33	2.62	2.84	3.54	3.88	4.36	4.73	1.75	0.85
	Unfastened	A	0.861	1.06	1.35	1.57	1.44	1.77	2.25	2.62	1.85	0.80
		B	1.75	2.00	2.35	2.62	2.91	3.33	3.92	4.37	1.65	0.90
		C	0.580	0.619	0.673	0.714	0.967	1.03	1.12	1.19	1.65	0.90
		D	0.894	1.03	1.22	1.36	1.49	1.71	2.03	2.27	1.90	0.80
10CS3.5x105	Fastened	A	1.99	2.42	3.03	3.50	3.32	4.04	5.05	5.83	1.75	0.85
		B	4.25	4.78	5.53	6.10	7.08	7.96	9.21	10.2	1.65	0.90
		C	1.82	2.03	2.31	2.54	3.04	3.38	3.86	4.23	1.75	0.85
		D	5.55	6.01	6.65	7.15	9.26	10.0	11.1	11.9	1.75	0.85
	Unfastened	A	1.99	2.42	3.03	3.50	3.32	4.04	5.05	5.83	1.85	0.80
		B	4.25	4.78	5.53	6.10	7.08	7.96	9.21	10.2	1.65	0.90
		C	1.94	2.05	2.20	2.32	3.24	3.41	3.67	3.86	1.65	0.90
		D	3.86	4.37	5.08	5.63	6.43	7.28	8.47	9.39	1.90	0.80
10CS3.5x085	Fastened	A	1.31	1.61	2.03	2.35	2.19	2.68	3.38	3.91	1.75	0.85
		B	2.70	3.07	3.58	3.97	4.50	5.11	5.96	6.62	1.65	0.90
		C	1.10	1.23	1.42	1.56	1.83	2.05	2.36	2.60	1.75	0.85
		D	3.48	3.79	4.23	4.57	5.80	6.32	7.05	7.62	1.75	0.85
	Unfastened	A	1.31	1.61	2.03	2.35	2.19	2.68	3.38	3.91	1.85	0.80
		B	2.70	3.07	3.58	3.97	4.50	5.11	5.96	6.62	1.65	0.90
		C	1.10	1.17	1.26	1.34	1.84	1.95	2.10	2.23	1.65	0.90
		D	1.95	2.23	2.62	2.91	3.25	3.71	4.36	4.86	1.90	0.80
10CS3.5x070	Fastened	A	0.889	1.10	1.39	1.62	1.48	1.83	2.32	2.70	1.75	0.85
		B	1.77	2.02	2.38	2.66	2.95	3.37	3.97	4.43	1.65	0.90
		C	0.676	0.763	0.887	0.982	1.13	1.27	1.48	1.64	1.75	0.85
		D	2.25	2.47	2.77	3.01	3.75	4.11	4.62	5.02	1.75	0.85
	Unfastened	A	0.889	1.10	1.39	1.62	1.48	1.83	2.32	2.70	1.85	0.80
		B	1.77	2.02	2.38	2.66	2.95	3.37	3.97	4.43	1.65	0.90
		C	0.636	0.678	0.737	0.783	1.06	1.13	1.23	1.30	1.65	0.90
		D	0.895	1.03	1.22	1.36	1.49	1.72	2.03	2.27	1.90	0.80

Table II - 13

Web Crippling, P_n, kips [1,2,4]
C-Sections With Lips



Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi				F_y = 55 ksi				Ω_w	ϕ_w
			Bearing Length, N, in.[4]				Bearing Length, N, in.[4]					
			1	2	4	6	1	2	4	6		
10CS3.5x065	Fastened	A	0.765	0.948	1.21	1.41	1.27	1.58	2.01	2.34	1.75	0.85
		B	1.50	1.73	2.04	2.28	2.51	2.88	3.40	3.80	1.65	0.90
		C	0.558	0.632	0.737	0.817	0.930	1.05	1.23	1.36	1.75	0.85
		D	1.90	2.09	2.36	2.56	3.17	3.48	3.93	4.27	1.75	0.85
	Unfastened	A	0.765	0.948	1.21	1.41	1.27	1.58	2.01	2.34	1.85	0.80
		B	1.50	1.73	2.04	2.28	2.51	2.88	3.40	3.80	1.65	0.90
		C	0.511	0.546	0.595	0.633	0.852	0.910	0.992	1.05	1.65	0.90
		D	0.613	0.708	0.841	0.943	1.02	1.18	1.40	1.57	1.90	0.80
8CS3.5x105	Fastened	A	2.05	2.49	3.11	3.59	3.41	4.15	5.18	5.98	1.75	0.85
		B	4.30	4.83	5.59	6.18	7.16	8.06	9.32	10.3	1.65	0.90
		C	1.99	2.22	2.53	2.77	3.32	3.69	4.22	4.62	1.75	0.85
		D	5.81	6.29	6.96	7.48	9.69	10.5	11.6	12.5	1.75	0.85
	Unfastened	A	2.05	2.49	3.11	3.59	3.41	4.15	5.18	5.98	1.85	0.80
		B	4.30	4.83	5.59	6.18	7.16	8.06	9.32	10.3	1.65	0.90
		C	2.07	2.19	2.35	2.48	3.46	3.65	3.92	4.13	1.65	0.90
		D	3.86	4.37	5.09	5.64	6.44	7.29	8.48	9.40	1.90	0.80
8CS3.5x085	Fastened	A	1.35	1.66	2.09	2.42	2.25	2.76	3.48	4.03	1.75	0.85
		B	2.74	3.11	3.63	4.03	4.56	5.18	6.04	6.71	1.65	0.90
		C	1.22	1.37	1.58	1.74	2.04	2.29	2.64	2.90	1.75	0.85
		D	3.67	4.00	4.46	4.82	6.12	6.67	7.44	8.03	1.75	0.85
	Unfastened	A	1.35	1.66	2.09	2.42	2.25	2.76	3.48	4.03	1.85	0.80
		B	2.74	3.11	3.63	4.03	4.56	5.18	6.04	6.71	1.65	0.90
		C	1.19	1.26	1.37	1.44	1.99	2.11	2.28	2.41	1.65	0.90
		D	1.95	2.23	2.62	2.92	3.26	3.72	4.36	4.86	1.90	0.80
8CS3.5x070	Fastened	A	0.919	1.14	1.44	1.68	1.53	1.89	2.40	2.79	1.75	0.85
		B	1.80	2.05	2.42	2.70	3.00	3.42	4.03	4.50	1.65	0.90
		C	0.772	0.871	1.01	1.12	1.29	1.45	1.69	1.87	1.75	0.85
		D	2.39	2.62	2.95	3.20	3.99	4.37	4.92	5.33	1.75	0.85
	Unfastened	A	0.919	1.14	1.44	1.68	1.53	1.89	2.40	2.79	1.85	0.80
		B	1.80	2.05	2.42	2.70	3.00	3.42	4.03	4.50	1.65	0.90
		C	0.698	0.744	0.809	0.859	1.16	1.24	1.35	1.43	1.65	0.90
		D	0.896	1.03	1.22	1.37	1.49	1.72	2.03	2.28	1.90	0.80
8CS3.5x065	Fastened	A	0.792	0.982	1.25	1.46	1.32	1.64	2.08	2.43	1.75	0.85
		B	1.53	1.75	2.07	2.31	2.55	2.92	3.45	3.85	1.65	0.90
		C	0.644	0.729	0.850	0.943	1.07	1.22	1.42	1.57	1.75	0.85
		D	2.03	2.23	2.51	2.73	3.38	3.72	4.19	4.55	1.75	0.85
	Unfastened	A	0.792	0.982	1.25	1.46	1.32	1.64	2.08	2.43	1.85	0.80
		B	1.53	1.75	2.07	2.31	2.55	2.92	3.45	3.85	1.65	0.90
		C	0.565	0.603	0.657	0.699	0.941	1.00	1.10	1.16	1.65	0.90
		D	0.614	0.708	0.842	0.944	1.02	1.18	1.40	1.57	1.90	0.80
8CS3.5x059	Fastened	A	0.652	0.811	1.04	1.21	1.09	1.35	1.73	2.02	1.75	0.85
		B	1.23	1.42	1.68	1.89	2.05	2.36	2.80	3.14	1.65	0.90
		C	0.506	0.575	0.673	0.749	0.843	0.959	1.12	1.25	1.75	0.85
		D	1.63	1.80	2.04	2.22	2.72	3.00	3.39	3.70	1.75	0.85
	Unfastened	A	0.652	0.811	1.04	1.21	1.09	1.35	1.73	2.02	1.85	0.80
		B	1.23	1.42	1.68	1.89	2.05	2.36	2.80	3.14	1.65	0.90
		C	0.424	0.455	0.497	0.530	0.707	0.758	0.828	0.883	1.65	0.90
		D	0.322	0.373	0.445	0.500	0.536	0.621	0.741	0.833	1.90	0.80

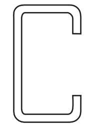
Table II - 13

Web Crippling, P_n, kips[1,2,4]
C-Sections With Lips

Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi, Bearing Length, N, in.[4] 1	2	4	6	F_y = 55 ksi, Bearing Length, N, in.[4] 1	2	4	6	Ω_w	ϕ_w
8CS2.5x105	Fastened	A	2.05	2.49	3.11	3.59	3.41	4.15	5.18	5.98	1.75	0.85
		B	4.30	4.83	5.59	6.18	7.16	8.06	9.32	10.3	1.65	0.90
		C	1.99	2.22	2.53	2.77	3.32	3.69	4.22	4.62	1.75	0.85
		D	5.81	6.29	6.96	7.48	9.69	10.5	11.6	12.5	1.75	0.85
	Unfastened	A	2.05	2.49	3.11	3.59	3.41	4.15	5.18	5.98	1.85	0.80
		B	4.30	4.83	5.59	6.18	7.16	8.06	9.32	10.3	1.65	0.90
		C	2.07	2.19	2.35	2.48	3.46	3.65	3.92	4.13	1.65	0.90
		D	3.86	4.37	5.09	5.64	6.44	7.29	8.48	9.40	1.90	0.80
8CS2.5x085	Fastened	A	1.35	1.66	2.09	2.42	2.25	2.76	3.48	4.03	1.75	0.85
		B	2.74	3.11	3.63	4.03	4.56	5.18	6.04	6.71	1.65	0.90
		C	1.22	1.37	1.58	1.74	2.04	2.29	2.64	2.90	1.75	0.85
		D	3.67	4.00	4.46	4.82	6.12	6.67	7.44	8.03	1.75	0.85
	Unfastened	A	1.35	1.66	2.09	2.42	2.25	2.76	3.48	4.03	1.85	0.80
		B	2.74	3.11	3.63	4.03	4.56	5.18	6.04	6.71	1.65	0.90
		C	1.19	1.26	1.37	1.44	1.99	2.11	2.28	2.41	1.65	0.90
		D	1.95	2.23	2.62	2.92	3.26	3.72	4.36	4.86	1.90	0.80
8CS2.5x070	Fastened	A	0.919	1.14	1.44	1.68	1.53	1.89	2.40	2.79	1.75	0.85
		B	1.80	2.05	2.42	2.70	3.00	3.42	4.03	4.50	1.65	0.90
		C	0.772	0.871	1.01	1.12	1.29	1.45	1.69	1.87	1.75	0.85
		D	2.39	2.62	2.95	3.20	3.99	4.37	4.92	5.33	1.75	0.85
	Unfastened	A	0.919	1.14	1.44	1.68	1.53	1.89	2.40	2.79	1.85	0.80
		B	1.80	2.05	2.42	2.70	3.00	3.42	4.03	4.50	1.65	0.90
		C	0.698	0.744	0.809	0.859	1.16	1.24	1.35	1.43	1.65	0.90
		D	0.896	1.03	1.22	1.37	1.49	1.72	2.03	2.28	1.90	0.80
8CS2.5x065	Fastened	A	0.792	0.982	1.25	1.46	1.32	1.64	2.08	2.43	1.75	0.85
		B	1.53	1.75	2.07	2.31	2.55	2.92	3.45	3.85	1.65	0.90
		C	0.644	0.729	0.850	0.943	1.07	1.22	1.42	1.57	1.75	0.85
		D	2.03	2.23	2.51	2.73	3.38	3.72	4.19	4.55	1.75	0.85
	Unfastened	A	0.792	0.982	1.25	1.46	1.32	1.64	2.08	2.43	1.85	0.80
		B	1.53	1.75	2.07	2.31	2.55	2.92	3.45	3.85	1.65	0.90
		C	0.565	0.603	0.657	0.699	0.941	1.00	1.10	1.16	1.65	0.90
		D	0.614	0.708	0.842	0.944	1.02	1.18	1.40	1.57	1.90	0.80
8CS2.5x059	Fastened	A	0.652	0.811	1.04	1.21	1.09	1.35	1.73	2.02	1.75	0.85
		B	1.23	1.42	1.68	1.89	2.05	2.36	2.80	3.14	1.65	0.90
		C	0.506	0.575	0.673	0.749	0.843	0.959	1.12	1.25	1.75	0.85
		D	1.63	1.80	2.04	2.22	2.72	3.00	3.39	3.70	1.75	0.85
	Unfastened	A	0.652	0.811	1.04	1.21	1.09	1.35	1.73	2.02	1.85	0.80
		B	1.23	1.42	1.68	1.89	2.05	2.36	2.80	3.14	1.65	0.90
		C	0.424	0.455	0.497	0.530	0.707	0.758	0.828	0.883	1.65	0.90
		D	0.322	0.373	0.445	0.500	0.536	0.621	0.741	0.833	1.90	0.80
6CS2.5x105	Fastened	A	2.11	2.56	3.20	3.69	3.51	4.27	5.34	6.16	1.75	0.85
		B	4.35	4.90	5.67	6.26	7.26	8.16	9.45	10.4	1.65	0.90
		C	2.19	2.43	2.78	3.05	3.65	4.06	4.63	5.08	1.75	0.85
		D	6.11	6.61	7.32	7.86	10.2	11.0	12.2	13.1	1.75	0.85
	Unfastened	A	2.11	2.56	3.20	3.69	3.51	4.27	5.34	6.16	1.85	0.80
		B	4.35	4.90	5.67	6.26	7.26	8.16	9.45	10.4	1.65	0.90
		C	2.23	2.35	2.53	2.66	3.71	3.92	4.21	4.43	1.65	0.90
		D	3.87	4.38	5.09	5.64	6.45	7.29	8.49	9.41	1.90	0.80

Table II - 13

Web Crippling, P_n, kips[1,2,4]
C-Sections With Lips



Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi, Bearing Length, N, in.[4]				F_y = 55 ksi, Bearing Length, N, in.[4]				Ω_w	ϕ_w
			1	2	4	6	1	2	4	6		
6CS2.5x085	Fastened	A	1.40	1.71	2.16	2.50	2.33	2.85	3.60	4.17	1.75	0.85
		B	2.78	3.15	3.68	4.09	4.63	5.25	6.13	6.81	1.65	0.90
		C	1.37	1.53	1.77	1.95	2.28	2.56	2.95	3.25	1.75	0.85
		D	3.89	4.24	4.73	5.10	6.48	7.06	7.88	8.51	1.75	0.85
	Unfastened	A	1.40	1.71	2.16	2.50	2.33	2.85	3.60	4.17	1.85	0.80
		B	2.78	3.15	3.68	4.09	4.63	5.25	6.13	6.81	1.65	0.90
		C	1.29	1.37	1.48	1.57	2.16	2.29	2.47	2.62	1.65	0.90
		D	1.96	2.23	2.62	2.92	3.26	3.72	4.37	4.87	1.90	0.80
6CS2.5x070	Fastened	A	0.953	1.18	1.50	1.74	1.59	1.96	2.49	2.90	1.75	0.85
		B	1.83	2.09	2.46	2.74	3.05	3.48	4.10	4.57	1.65	0.90
		C	0.881	0.995	1.16	1.28	1.47	1.66	1.93	2.13	1.75	0.85
		D	2.56	2.80	3.15	3.42	4.26	4.67	5.25	5.69	1.75	0.85
	Unfastened	A	0.953	1.18	1.50	1.74	1.59	1.96	2.49	2.90	1.85	0.80
		B	1.83	2.09	2.46	2.74	3.05	3.48	4.10	4.57	1.65	0.90
		C	0.769	0.820	0.891	0.946	1.28	1.37	1.49	1.58	1.65	0.90
		D	0.898	1.03	1.22	1.37	1.50	1.72	2.04	2.28	1.90	0.80
6CS2.5x065	Fastened	A	0.823	1.02	1.30	1.51	1.37	1.70	2.17	2.52	1.75	0.85
		B	1.55	1.78	2.10	2.35	2.59	2.97	3.51	3.92	1.65	0.90
		C	0.742	0.841	0.980	1.09	1.24	1.40	1.63	1.81	1.75	0.85
		D	2.17	2.39	2.69	2.93	3.62	3.98	4.49	4.88	1.75	0.85
	Unfastened	A	0.823	1.02	1.30	1.51	1.37	1.70	2.17	2.52	1.85	0.80
		B	1.55	1.78	2.10	2.35	2.59	2.97	3.51	3.92	1.65	0.90
		C	0.626	0.668	0.728	0.774	1.04	1.11	1.21	1.29	1.65	0.90
		D	0.615	0.710	0.843	0.946	1.03	1.18	1.41	1.58	1.90	0.80
6CS2.5x059	Fastened	A	0.679	0.845	1.08	1.26	1.13	1.41	1.80	2.10	1.75	0.85
		B	1.25	1.44	1.71	1.92	2.09	2.41	2.86	3.20	1.65	0.90
		C	0.592	0.673	0.788	0.875	0.986	1.12	1.31	1.46	1.75	0.85
		D	1.76	1.94	2.19	2.39	2.93	3.23	3.66	3.98	1.75	0.85
	Unfastened	A	0.679	0.845	1.08	1.26	1.13	1.41	1.80	2.10	1.85	0.80
		B	1.25	1.44	1.71	1.92	2.09	2.41	2.86	3.20	1.65	0.90
		C	0.475	0.508	0.556	0.592	0.791	0.847	0.926	0.987	1.65	0.90
		D	0.322	0.373	0.446	0.501	0.537	0.622	0.743	0.835	1.90	0.80
4CS2.5x105	Fastened	A	2.18	2.65	3.31	3.82	3.63	4.42	5.52	6.37	1.75	0.85
		B	4.42	4.98	5.76	6.36	7.37	8.29	9.60	10.6	1.65	0.90
		C	2.43	2.70	3.08	3.38	4.04	4.49	5.13	5.63	1.75	0.85
		D	6.47	7.00	7.75	8.33	10.8	11.7	12.9	13.9	1.75	0.85
	Unfastened	A	2.18	2.65	3.31	3.82	3.63	4.42	5.52	6.37	1.85	0.80
		B	4.42	4.98	5.76	6.36	7.37	8.29	9.60	10.6	1.65	0.90
		C	2.41	2.55	2.73	2.88	4.02	4.24	4.56	4.80	1.65	0.90
		D	3.88	4.38	5.10	5.65	6.46	7.31	8.50	9.42	1.90	0.80
4CS2.5x085	Fastened	A	1.45	1.78	2.24	2.60	2.42	2.96	3.74	4.33	1.75	0.85
		B	2.83	3.21	3.75	4.16	4.71	5.35	6.24	6.93	1.65	0.90
		C	1.54	1.73	1.99	2.20	2.57	2.88	3.32	3.66	1.75	0.85
		D	4.15	4.52	5.05	5.45	6.92	7.54	8.41	9.08	1.75	0.85
	Unfastened	A	1.45	1.78	2.24	2.60	2.42	2.96	3.74	4.33	1.85	0.80
		B	2.83	3.21	3.75	4.16	4.71	5.35	6.24	6.93	1.65	0.90
		C	1.42	1.51	1.63	1.72	2.37	2.51	2.71	2.87	1.65	0.90
		D	1.96	2.24	2.63	2.93	3.27	3.73	4.38	4.88	1.90	0.80

Table II - 13

Web Crippling, P_n, kips [1,2,4]
C-Sections With Lips

Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi				F_y = 55 ksi				Ω_w	ϕ_w
			Bearing Length, N, in.[4]				Bearing Length, N, in.[4]					
			1	2	4	6	1	2	4	6		
4CS2.5x070	Fastened	A	0.995	1.23	1.56	1.82	1.66	2.05	2.60	3.03	1.75	0.85
		B	1.86	2.13	2.51	2.80	3.11	3.55	4.18	4.66	1.65	0.90
		C	1.01	1.14	1.33	1.47	1.69	1.91	2.22	2.45	1.75	0.85
		D	2.75	3.02	3.39	3.68	4.59	5.03	5.65	6.13	1.75	0.85
	Unfastened	A	0.995	1.23	1.56	1.82	1.66	2.05	2.60	3.03	1.85	0.80
		B	1.86	2.13	2.51	2.80	3.11	3.55	4.18	4.66	1.65	0.90
		C	0.854	0.911	0.990	1.05	1.42	1.52	1.65	1.75	1.65	0.90
		D	0.899	1.03	1.22	1.37	1.50	1.72	2.04	2.29	1.90	0.80
4CS2.5x065	Fastened	A	0.861	1.07	1.36	1.58	1.43	1.78	2.26	2.64	1.75	0.85
		B	1.59	1.82	2.15	2.40	2.64	3.03	3.58	4.00	1.65	0.90
		C	0.861	0.975	1.14	1.26	1.44	1.63	1.89	2.10	1.75	0.85
		D	2.35	2.58	2.91	3.16	3.92	4.30	4.85	5.27	1.75	0.85
	Unfastened	A	0.861	1.07	1.36	1.58	1.43	1.78	2.26	2.64	1.85	0.80
		B	1.59	1.82	2.15	2.40	2.64	3.03	3.58	4.00	1.65	0.90
		C	0.699	0.747	0.814	0.866	1.17	1.24	1.36	1.44	1.65	0.90
		D	0.616	0.711	0.845	0.947	1.03	1.18	1.41	1.58	1.90	0.80
4CS2.5x059	Fastened	A	0.712	0.886	1.13	1.32	1.19	1.48	1.89	2.20	1.75	0.85
		B	1.28	1.48	1.75	1.96	2.14	2.46	2.92	3.27	1.65	0.90
		C	0.695	0.790	0.925	1.03	1.16	1.32	1.54	1.71	1.75	0.85
		D	1.91	2.11	2.38	2.60	3.18	3.51	3.97	4.33	1.75	0.85
	Unfastened	A	0.712	0.886	1.13	1.32	1.19	1.48	1.89	2.20	1.85	0.80
		B	1.28	1.48	1.75	1.96	2.14	2.46	2.92	3.27	1.65	0.90
		C	0.535	0.573	0.626	0.667	0.892	0.955	1.04	1.11	1.65	0.90
		D	0.323	0.374	0.446	0.502	0.538	0.624	0.744	0.836	1.90	0.80

Notes:

1. Web crippling strengths are nominal strengths calculated without consideration of holes or other openings. To obtain available strengths, the values must be modified by safety factors (ASD) or resistance factors (LRFD), which are provided in the table.
2. Strength reductions factors for openings must be calculated in accordance with the provisions of *Specification* Section C3.4.2.
3. Case A
   End Reaction, Opposing Loads Spaced > 1.5h

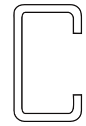
   Case B
   Interior Reactions, Opposing Loads Spaced > 1.5h

   Case C
   End Reaction, Opposing Loads Spaced ≤ 1.5h

   Case D
   Interior Reactions, Opposing Loads Spaced ≤ 1.5h
4. Linear interpolation is permitted between bearing lengths.

Table II - 14

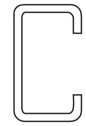
Web Crippling, P_n, kips [1,2,4]
SSMA Studs
C-Sections With Lips



Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi				F_y = 50 ksi				Ω_w	ϕ_w
			Bearing Length, N, in.[4]				Bearing Length, N, in.[4]					
			1	2	4	6	1	2	4	6		
1200S200-97	Fastened	A	1.87	2.27	2.85	3.29	2.83	3.44	4.31	4.98	1.75	0.85
		B	4.10	4.62	5.35	5.91	6.21	6.99	8.10	8.96	1.65	0.90
		C	1.56	1.73	1.98	2.17	2.36	2.63	3.00	3.29	1.75	0.85
		D	5.02	5.44	6.03	6.48	7.61	8.24	9.14	9.82	1.75	0.85
	Unfastened	A	1.87	2.27	2.85	3.29	2.83	3.44	4.31	4.98	1.85	0.80
		B	4.10	4.62	5.35	5.91	6.21	6.99	8.10	8.96	1.65	0.90
		C	1.79	1.89	2.04	2.15	2.72	2.87	3.09	3.25	1.65	0.90
		D	4.33	4.90	5.71	6.33	6.55	7.42	8.65	9.59	1.90	0.80
1200S200-68	Fastened	A	0.956	1.18	1.50	1.74	1.45	1.79	2.27	2.64	1.75	0.85
		B	2.08	2.38	2.80	3.12	3.16	3.60	4.24	4.73	1.65	0.90
		C	0.636	0.718	0.833	0.922	0.964	1.09	1.26	1.40	1.75	0.85
		D	2.31	2.53	2.84	3.08	3.50	3.84	4.31	4.67	1.75	0.85
	Unfastened	A	0.956	1.18	1.50	1.74	1.45	1.79	2.27	2.64	1.85	0.80
		B	2.08	2.38	2.80	3.12	3.16	3.60	4.24	4.73	1.65	0.90
		C	0.770	0.820	0.891	0.945	1.17	1.24	1.35	1.43	1.65	0.90
		D	2.25	2.59	3.06	3.43	3.41	3.92	4.64	5.19	1.90	0.80
1000S250-97	Fastened	A	1.91	2.33	2.92	3.37	2.90	3.53	4.42	5.10	1.75	0.85
		B	4.14	4.67	5.41	5.98	6.28	7.07	8.19	9.05	1.65	0.90
		C	1.70	1.90	2.17	2.38	2.58	2.87	3.29	3.61	1.75	0.85
		D	5.25	5.68	6.30	6.77	7.95	8.61	9.55	10.3	1.75	0.85
	Unfastened	A	1.91	2.33	2.92	3.37	2.90	3.53	4.42	5.10	1.85	0.80
		B	4.14	4.67	5.41	5.98	6.28	7.07	8.19	9.05	1.65	0.90
		C	1.91	2.02	2.17	2.29	2.90	3.06	3.29	3.47	1.65	0.90
		D	4.33	4.90	5.72	6.34	6.56	7.43	8.66	9.60	1.90	0.80
1000S250-68	Fastened	A	0.986	1.22	1.55	1.80	1.49	1.84	2.34	2.72	1.75	0.85
		B	2.11	2.41	2.84	3.16	3.20	3.65	4.30	4.79	1.65	0.90
		C	0.727	0.820	0.952	1.05	1.10	1.24	1.44	1.60	1.75	0.85
		D	2.45	2.68	3.01	3.26	3.71	4.06	4.56	4.95	1.75	0.85
	Unfastened	A	0.986	1.22	1.55	1.80	1.49	1.84	2.34	2.72	1.85	0.80
		B	2.11	2.41	2.84	3.16	3.20	3.65	4.30	4.79	1.65	0.90
		C	0.842	0.897	0.975	1.03	1.28	1.36	1.48	1.57	1.65	0.90
		D	2.26	2.59	3.07	3.43	3.42	3.93	4.65	5.20	1.90	0.80
1000S250-54	Fastened	A	0.639	0.796	1.02	1.19	0.968	1.21	1.54	1.80	1.75	0.85
		B	1.36	1.57	1.87	2.09	2.06	2.38	2.83	3.17	1.65	0.90
		C	0.399	0.455	0.533	0.593	0.605	0.689	0.808	0.899	1.75	0.85
		D	1.47	1.63	1.84	2.01	2.23	2.46	2.79	3.04	1.75	0.85
	Unfastened	A	0.639	0.796	1.02	1.19	0.968	1.21	1.54	1.80	1.85	0.80
		B	1.36	1.57	1.87	2.09	2.06	2.38	2.83	3.17	1.65	0.90
		C	0.481	0.516	0.565	0.602	0.729	0.782	0.856	0.913	1.65	0.90
		D	1.48	1.72	2.06	2.31	2.25	2.61	3.11	3.51	1.90	0.80
1000S200-97	Fastened	A	1.91	2.33	2.92	3.37	2.90	3.53	4.42	5.10	1.75	0.85
		B	4.14	4.67	5.41	5.98	6.28	7.07	8.19	9.05	1.65	0.90
		C	1.70	1.90	2.17	2.38	2.58	2.87	3.29	3.61	1.75	0.85
		D	5.25	5.68	6.30	6.77	7.95	8.61	9.55	10.3	1.75	0.85
	Unfastened	A	1.91	2.33	2.92	3.37	2.90	3.53	4.42	5.10	1.85	0.80
		B	4.14	4.67	5.41	5.98	6.28	7.07	8.19	9.05	1.65	0.90
		C	1.91	2.02	2.17	2.29	2.90	3.06	3.29	3.47	1.65	0.90
		D	4.33	4.90	5.72	6.34	6.56	7.43	8.66	9.60	1.90	0.80

Table II - 14

Web Crippling, P_n, kips [1,2,4]
SSMA Studs
C-Sections With Lips



Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi				F_y = 50 ksi				Ω_w	ϕ_w
			Bearing Length, N, in.[4]				Bearing Length, N, in.[4]					
			1	2	4	6	1	2	4	6		
1000S200-68	Fastened	A	0.986	1.22	1.55	1.80	1.49	1.84	2.34	2.72	1.75	0.85
		B	2.11	2.41	2.84	3.16	3.20	3.65	4.30	4.79	1.65	0.90
		C	0.727	0.820	0.952	1.05	1.10	1.24	1.44	1.60	1.75	0.85
		D	2.45	2.68	3.01	3.26	3.71	4.06	4.56	4.95	1.75	0.85
	Unfastened	A	0.986	1.22	1.55	1.80	1.49	1.84	2.34	2.72	1.85	0.80
		B	2.11	2.41	2.84	3.16	3.20	3.65	4.30	4.79	1.65	0.90
		C	0.842	0.897	0.975	1.03	1.28	1.36	1.48	1.57	1.65	0.90
		D	2.26	2.59	3.07	3.43	3.42	3.93	4.65	5.20	1.90	0.80
1000S200-54	Fastened	A	0.639	0.796	1.02	1.19	0.968	1.21	1.54	1.80	1.75	0.85
		B	1.36	1.57	1.87	2.09	2.06	2.38	2.83	3.17	1.65	0.90
		C	0.399	0.455	0.533	0.593	0.605	0.689	0.808	0.899	1.75	0.85
		D	1.47	1.63	1.84	2.01	2.23	2.46	2.79	3.04	1.75	0.85
	Unfastened	A	0.639	0.796	1.02	1.19	0.968	1.21	1.54	1.80	1.85	0.80
		B	1.36	1.57	1.87	2.09	2.06	2.38	2.83	3.17	1.65	0.90
		C	0.481	0.516	0.565	0.602	0.729	0.782	0.856	0.913	1.65	0.90
		D	1.48	1.72	2.06	2.31	2.25	2.61	3.11	3.51	1.90	0.80
800S200-97	Fastened	A	1.97	2.39	2.99	3.46	2.98	3.62	4.54	5.24	1.75	0.85
		B	4.19	4.72	5.47	6.05	6.35	7.15	8.29	9.16	1.65	0.90
		C	1.87	2.08	2.38	2.61	2.83	3.15	3.61	3.95	1.75	0.85
		D	5.50	5.96	6.60	7.10	8.33	9.02	10.0	10.8	1.75	0.85
	Unfastened	A	1.97	2.39	2.99	3.46	2.98	3.62	4.54	5.24	1.85	0.80
		B	4.19	4.72	5.47	6.05	6.35	7.15	8.29	9.16	1.65	0.90
		C	2.05	2.16	2.33	2.45	3.11	3.28	3.53	3.72	1.65	0.90
		D	4.33	4.91	5.72	6.35	6.57	7.44	8.67	9.61	1.90	0.80
800S200-68	Fastened	A	1.02	1.26	1.60	1.86	1.54	1.91	2.42	2.81	1.75	0.85
		B	2.14	2.45	2.88	3.21	3.24	3.71	4.36	4.86	1.65	0.90
		C	0.827	0.934	1.08	1.20	1.25	1.41	1.64	1.82	1.75	0.85
		D	2.60	2.85	3.20	3.47	3.94	4.31	4.84	5.25	1.75	0.85
	Unfastened	A	1.02	1.26	1.60	1.86	1.54	1.91	2.42	2.81	1.85	0.80
		B	2.14	2.45	2.88	3.21	3.24	3.71	4.36	4.86	1.65	0.90
		C	0.922	0.983	1.07	1.13	1.40	1.49	1.62	1.72	1.65	0.90
		D	2.26	2.60	3.07	3.44	3.42	3.93	4.65	5.21	1.90	0.80
800S200-54	Fastened	A	0.664	0.827	1.06	1.24	1.01	1.25	1.60	1.87	1.75	0.85
		B	1.39	1.60	1.90	2.13	2.10	2.42	2.88	3.23	1.65	0.90
		C	0.473	0.539	0.631	0.703	0.717	0.816	0.957	1.06	1.75	0.85
		D	1.58	1.75	1.98	2.16	2.40	2.65	3.00	3.27	1.75	0.85
	Unfastened	A	0.664	0.827	1.06	1.24	1.01	1.25	1.60	1.87	1.85	0.80
		B	1.39	1.60	1.90	2.13	2.10	2.42	2.88	3.23	1.65	0.90
		C	0.539	0.578	0.633	0.675	0.817	0.875	0.958	1.02	1.65	0.90
		D	1.48	1.72	2.06	2.32	2.25	2.61	3.12	3.51	1.90	0.80
800S200-43	Fastened	A	0.432	0.544	0.701	0.822	-	-	-	-	1.75	0.85
		B	0.894	1.04	1.25	1.41	-	-	-	-	1.65	0.90
		C	0.262	0.301	0.357	0.400	-	-	-	-	1.75	0.85
		D	0.959	1.07	1.22	1.34	-	-	-	-	1.75	0.85
	Unfastened	A	0.432	0.544	0.701	0.822	-	-	-	-	1.85	0.80
		B	0.894	1.04	1.25	1.41	-	-	-	-	1.65	0.90
		C	0.306	0.330	0.365	0.391	-	-	-	-	1.65	0.90
		D	0.940	1.10	1.33	1.50	-	-	-	-	1.90	0.80

Table II - 14

Web Crippling, P_n, kips [1,2,4]
SSMA Studs
C-Sections With Lips

Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi				F_y = 50 ksi				Ω_w	ϕ_w
			Bearing Length, N, in.[4]				Bearing Length, N, in.[4]					
			1	2	4	6	1	2	4	6		
800S162-97	Fastened	A	1.97	2.39	2.99	3.46	2.98	3.62	4.54	5.24	1.75	0.85
		B	4.19	4.72	5.47	6.05	6.35	7.15	8.29	9.16	1.65	0.90
		C	1.87	2.08	2.38	2.61	2.83	3.15	3.61	3.95	1.75	0.85
		D	5.50	5.96	6.60	7.10	8.33	9.02	10.0	10.8	1.75	0.85
	Unfastened	A	1.97	2.39	2.99	3.46	2.98	3.62	4.54	5.24	1.85	0.80
		B	4.19	4.72	5.47	6.05	6.35	7.15	8.29	9.16	1.65	0.90
		C	2.05	2.16	2.33	2.45	3.11	3.28	3.53	3.72	1.65	0.90
		D	4.33	4.91	5.72	6.35	6.57	7.44	8.67	9.61	1.90	0.80
800S162-68	Fastened	A	1.02	1.26	1.60	1.86	1.54	1.91	2.42	2.81	1.75	0.85
		B	2.14	2.45	2.88	3.21	3.24	3.71	4.36	4.86	1.65	0.90
		C	0.827	0.934	1.08	1.20	1.25	1.41	1.64	1.82	1.75	0.85
		D	2.60	2.85	3.20	3.47	3.94	4.31	4.84	5.25	1.75	0.85
	Unfastened	A	1.02	1.26	1.60	1.86	1.54	1.91	2.42	2.81	1.85	0.80
		B	2.14	2.45	2.88	3.21	3.24	3.71	4.36	4.86	1.65	0.90
		C	0.922	0.983	1.07	1.13	1.40	1.49	1.62	1.72	1.65	0.90
		D	2.26	2.60	3.07	3.44	3.42	3.93	4.65	5.21	1.90	0.80
800S162-54	Fastened	A	0.664	0.827	1.06	1.24	1.01	1.25	1.60	1.87	1.75	0.85
		B	1.39	1.60	1.90	2.13	2.10	2.42	2.88	3.23	1.65	0.90
		C	0.473	0.539	0.631	0.703	0.717	0.816	0.957	1.06	1.75	0.85
		D	1.58	1.75	1.98	2.16	2.40	2.65	3.00	3.27	1.75	0.85
	Unfastened	A	0.664	0.827	1.06	1.24	1.01	1.25	1.60	1.87	1.85	0.80
		B	1.39	1.60	1.90	2.13	2.10	2.42	2.88	3.23	1.65	0.90
		C	0.539	0.578	0.633	0.675	0.817	0.875	0.958	1.02	1.65	0.90
		D	1.48	1.72	2.06	2.32	2.25	2.61	3.12	3.51	1.90	0.80
800S162-43	Fastened	A	0.432	0.544	0.701	0.822	-	-	-	-	1.75	0.85
		B	0.894	1.04	1.25	1.41	-	-	-	-	1.65	0.90
		C	0.262	0.301	0.357	0.400	-	-	-	-	1.75	0.85
		D	0.959	1.07	1.22	1.34	-	-	-	-	1.75	0.85
	Unfastened	A	0.432	0.544	0.701	0.822	-	-	-	-	1.85	0.80
		B	0.894	1.04	1.25	1.41	-	-	-	-	1.65	0.90
		C	0.306	0.330	0.365	0.391	-	-	-	-	1.65	0.90
		D	0.940	1.10	1.33	1.50	-	-	-	-	1.90	0.80
600S200-97	Fastened	A	2.02	2.46	3.08	3.56	3.07	3.73	4.67	5.39	1.75	0.85
		B	4.25	4.79	5.55	6.13	6.44	7.25	8.40	9.28	1.65	0.90
		C	2.06	2.29	2.62	2.87	3.12	3.47	3.97	4.35	1.75	0.85
		D	5.79	6.27	6.95	7.47	8.77	9.49	10.5	11.3	1.75	0.85
	Unfastened	A	2.02	2.46	3.08	3.56	3.07	3.73	4.67	5.39	1.85	0.80
		B	4.25	4.79	5.55	6.13	6.44	7.25	8.40	9.28	1.65	0.90
		C	2.20	2.33	2.50	2.64	3.34	3.53	3.79	3.99	1.65	0.90
		D	4.34	4.92	5.73	6.35	6.58	7.45	8.68	9.63	1.90	0.80
600S200-68	Fastened	A	1.06	1.30	1.66	1.92	1.60	1.98	2.51	2.92	1.75	0.85
		B	2.18	2.49	2.92	3.26	3.30	3.77	4.43	4.94	1.65	0.90
		C	0.942	1.06	1.23	1.37	1.43	1.61	1.87	2.07	1.75	0.85
		D	2.77	3.04	3.41	3.70	4.20	4.60	5.17	5.60	1.75	0.85
	Unfastened	A	1.06	1.30	1.66	1.92	1.60	1.98	2.51	2.92	1.85	0.80
		B	2.18	2.49	2.92	3.26	3.30	3.77	4.43	4.94	1.65	0.90
		C	1.01	1.08	1.17	1.25	1.54	1.64	1.78	1.89	1.65	0.90
		D	2.26	2.60	3.08	3.44	3.43	3.94	4.66	5.21	1.90	0.80

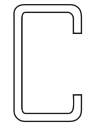
Table II - 14

Web Crippling, P_n, kips [1,2,4]
SSMA Studs
C-Sections With Lips

Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi				F_y = 50 ksi				Ω_w	ϕ_w
			Bearing Length, N, in.[4]				Bearing Length, N, in.[4]					
			1	2	4	6	1	2	4	6		
600S200-54	Fastened	A	0.692	0.862	1.10	1.29	1.05	1.31	1.67	1.95	1.75	0.85
		B	1.41	1.63	1.93	2.17	2.14	2.47	2.93	3.28	1.65	0.90
		C	0.557	0.634	0.744	0.828	0.844	0.961	1.13	1.25	1.75	0.85
		D	1.71	1.88	2.14	2.33	2.59	2.86	3.24	3.53	1.75	0.85
	Unfastened	A	0.692	0.862	1.10	1.29	1.05	1.31	1.67	1.95	1.85	0.80
		B	1.41	1.63	1.93	2.17	2.14	2.47	2.93	3.28	1.65	0.90
		C	0.605	0.648	0.710	0.757	0.916	0.982	1.08	1.15	1.65	0.90
		D	1.49	1.73	2.06	2.32	2.25	2.61	3.12	3.52	1.90	0.80
600S200-43	Fastened	A	0.453	0.570	0.736	0.862	-	-	-	-	1.75	0.85
		B	0.913	1.06	1.28	1.44	-	-	-	-	1.65	0.90
		C	0.324	0.372	0.441	0.494	-	-	-	-	1.75	0.85
		D	1.05	1.17	1.34	1.47	-	-	-	-	1.75	0.85
	Unfastened	A	0.453	0.570	0.736	0.862	-	-	-	-	1.85	0.80
		B	0.913	1.06	1.28	1.44	-	-	-	-	1.65	0.90
		C	0.353	0.381	0.420	0.451	-	-	-	-	1.65	0.90
		D	0.942	1.10	1.33	1.51	-	-	-	-	1.90	0.80
600S200-33	Fastened	A	0.267	0.340	0.442	0.520	-	-	-	-	1.75	0.85
		B	0.516	0.607	0.737	0.837	-	-	-	-	1.65	0.90
		C	0.163	0.190	0.227	0.256	-	-	-	-	1.75	0.85
		D	0.577	0.648	0.750	0.828	-	-	-	-	1.75	0.85
	Unfastened	A	0.267	0.340	0.442	0.520	-	-	-	-	1.85	0.80
		B	0.516	0.607	0.737	0.837	-	-	-	-	1.65	0.90
		C	0.165	0.179	0.200	0.216	-	-	-	-	1.65	0.90
		D	0.383	0.454	0.555	0.632	-	-	-	-	1.90	0.80
600S162-97	Fastened	A	2.02	2.46	3.08	3.56	3.07	3.73	4.67	5.39	1.75	0.85
		B	4.25	4.79	5.55	6.13	6.44	7.25	8.40	9.28	1.65	0.90
		C	2.06	2.29	2.62	2.87	3.12	3.47	3.97	4.35	1.75	0.85
		D	5.79	6.27	6.95	7.47	8.77	9.49	10.5	11.3	1.75	0.85
	Unfastened	A	2.02	2.46	3.08	3.56	3.07	3.73	4.67	5.39	1.85	0.80
		B	4.25	4.79	5.55	6.13	6.44	7.25	8.40	9.28	1.65	0.90
		C	2.20	2.33	2.50	2.64	3.34	3.53	3.79	3.99	1.65	0.90
		D	4.34	4.92	5.73	6.35	6.58	7.45	8.68	9.63	1.90	0.80
600S162-68	Fastened	A	1.06	1.30	1.66	1.92	1.60	1.98	2.51	2.92	1.75	0.85
		B	2.18	2.49	2.92	3.26	3.30	3.77	4.43	4.94	1.65	0.90
		C	0.942	1.06	1.23	1.37	1.43	1.61	1.87	2.07	1.75	0.85
		D	2.77	3.04	3.41	3.70	4.20	4.60	5.17	5.60	1.75	0.85
	Unfastened	A	1.06	1.30	1.66	1.92	1.60	1.98	2.51	2.92	1.85	0.80
		B	2.18	2.49	2.92	3.26	3.30	3.77	4.43	4.94	1.65	0.90
		C	1.01	1.08	1.17	1.25	1.54	1.64	1.78	1.89	1.65	0.90
		D	2.26	2.60	3.08	3.44	3.43	3.94	4.66	5.21	1.90	0.80
600S162-54	Fastened	A	0.692	0.862	1.10	1.29	1.05	1.31	1.67	1.95	1.75	0.85
		B	1.41	1.63	1.93	2.17	2.14	2.47	2.93	3.28	1.65	0.90
		C	0.557	0.634	0.744	0.828	0.844	0.961	1.13	1.25	1.75	0.85
		D	1.71	1.88	2.14	2.33	2.59	2.86	3.24	3.53	1.75	0.85
	Unfastened	A	0.692	0.862	1.10	1.29	1.05	1.31	1.67	1.95	1.85	0.80
		B	1.41	1.63	1.93	2.17	2.14	2.47	2.93	3.28	1.65	0.90
		C	0.605	0.648	0.710	0.757	0.916	0.982	1.08	1.15	1.65	0.90
		D	1.49	1.73	2.06	2.32	2.25	2.61	3.12	3.52	1.90	0.80

Table II - 14

Web Crippling, P_n, kips [1,2,4]
SSMA Studs
C-Sections With Lips



Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi				F_y = 50 ksi				Ω_w	ϕ_w
			Bearing Length, N, in.[4]				Bearing Length, N, in.[4]					
			1	2	4	6	1	2	4	6		
600S162-43	Fastened	A	0.453	0.570	0.736	0.862	-	-	-	-	1.75	0.85
		B	0.913	1.06	1.28	1.44	-	-	-	-	1.65	0.90
		C	0.324	0.372	0.441	0.494	-	-	-	-	1.75	0.85
		D	1.05	1.17	1.34	1.47	-	-	-	-	1.75	0.85
	Unfastened	A	0.453	0.570	0.736	0.862	-	-	-	-	1.85	0.80
		B	0.913	1.06	1.28	1.44	-	-	-	-	1.65	0.90
		C	0.353	0.381	0.420	0.451	-	-	-	-	1.65	0.90
		D	0.942	1.10	1.33	1.51	-	-	-	-	1.90	0.80
600S162-33	Fastened	A	0.267	0.340	0.442	0.520	-	-	-	-	1.75	0.85
		B	0.516	0.607	0.737	0.837	-	-	-	-	1.65	0.90
		C	0.163	0.190	0.227	0.256	-	-	-	-	1.75	0.85
		D	0.577	0.648	0.750	0.828	-	-	-	-	1.75	0.85
	Unfastened	A	0.267	0.340	0.442	0.520	-	-	-	-	1.85	0.80
		B	0.516	0.607	0.737	0.837	-	-	-	-	1.65	0.90
		C	0.165	0.179	0.200	0.216	-	-	-	-	1.65	0.90
		D	0.383	0.454	0.555	0.632	-	-	-	-	1.90	0.80
550S162-68	Fastened	A	1.07	1.32	1.67	1.94	1.62	2.00	2.53	2.94	1.75	0.85
		B	2.18	2.50	2.94	3.27	3.31	3.78	4.45	4.96	1.65	0.90
		C	0.974	1.10	1.28	1.41	1.48	1.67	1.93	2.14	1.75	0.85
		D	2.82	3.09	3.47	3.76	4.27	4.68	5.26	5.70	1.75	0.85
	Unfastened	A	1.07	1.32	1.67	1.94	1.62	2.00	2.53	2.94	1.85	0.80
		B	2.18	2.50	2.94	3.27	3.31	3.78	4.45	4.96	1.65	0.90
		C	1.04	1.11	1.20	1.28	1.57	1.68	1.82	1.93	1.65	0.90
		D	2.26	2.60	3.08	3.44	3.43	3.94	4.66	5.22	1.90	0.80
550S162-54	Fastened	A	0.700	0.872	1.12	1.30	1.06	1.32	1.69	1.97	1.75	0.85
		B	1.42	1.64	1.94	2.18	2.15	2.48	2.94	3.30	1.65	0.90
		C	0.580	0.661	0.775	0.862	0.879	1.00	1.17	1.31	1.75	0.85
		D	1.74	1.92	2.18	2.38	2.64	2.91	3.30	3.60	1.75	0.85
	Unfastened	A	0.700	0.872	1.12	1.30	1.06	1.32	1.69	1.97	1.85	0.80
		B	1.42	1.64	1.94	2.18	2.15	2.48	2.94	3.30	1.65	0.90
		C	0.623	0.668	0.731	0.780	0.944	1.01	1.11	1.18	1.65	0.90
		D	1.49	1.73	2.06	2.32	2.25	2.62	3.13	3.52	1.90	0.80
550S162-43	Fastened	A	0.459	0.578	0.745	0.874	-	-	-	-	1.75	0.85
		B	0.918	1.07	1.28	1.45	-	-	-	-	1.65	0.90
		C	0.341	0.392	0.464	0.520	-	-	-	-	1.75	0.85
		D	1.07	1.20	1.37	1.50	-	-	-	-	1.75	0.85
	Unfastened	A	0.459	0.578	0.745	0.874	-	-	-	-	1.85	0.80
		B	0.918	1.07	1.28	1.45	-	-	-	-	1.65	0.90
		C	0.366	0.395	0.436	0.467	-	-	-	-	1.65	0.90
		D	0.943	1.10	1.33	1.51	-	-	-	-	1.90	0.80
550S162-33	Fastened	A	0.272	0.345	0.449	0.528	-	-	-	-	1.75	0.85
		B	0.519	0.611	0.742	0.842	-	-	-	-	1.65	0.90
		C	0.175	0.203	0.243	0.274	-	-	-	-	1.75	0.85
		D	0.594	0.668	0.772	0.852	-	-	-	-	1.75	0.85
	Unfastened	A	0.272	0.345	0.449	0.528	-	-	-	-	1.85	0.80
		B	0.519	0.611	0.742	0.842	-	-	-	-	1.65	0.90
		C	0.173	0.188	0.209	0.226	-	-	-	-	1.65	0.90
		D	0.384	0.455	0.555	0.632	-	-	-	-	1.90	0.80

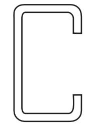
Table II - 14

Web Crippling, P_n, kips [1,2,4]
SSMA Studs
C-Sections With Lips

Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi				F_y = 50 ksi				Ω_w	ϕ_w
			Bearing Length, N, in.[4]				Bearing Length, N, in.[4]					
			1	2	4	6	1	2	4	6		
400S162-68	Fastened	A	1.10	1.36	1.73	2.01	1.67	2.06	2.61	3.04	1.75	0.85
		B	2.22	2.53	2.98	3.32	3.36	3.84	4.51	5.03	1.65	0.90
		C	1.08	1.22	1.42	1.57	1.64	1.85	2.14	2.37	1.75	0.85
		D	2.98	3.26	3.67	3.97	4.51	4.94	5.55	6.02	1.75	0.85
	Unfastened	A	1.10	1.36	1.73	2.01	1.67	2.06	2.61	3.04	1.85	0.80
		B	2.22	2.53	2.98	3.32	3.36	3.84	4.51	5.03	1.65	0.90
		C	1.12	1.20	1.30	1.38	1.70	1.81	1.97	2.09	1.65	0.90
		D	2.27	2.60	3.08	3.45	3.43	3.95	4.67	5.22	1.90	0.80
400S162-54	Fastened	A	0.725	0.904	1.16	1.35	1.10	1.37	1.75	2.05	1.75	0.85
		B	1.44	1.66	1.97	2.21	2.18	2.52	2.99	3.36	1.65	0.90
		C	0.657	0.749	0.878	0.977	0.996	1.13	1.33	1.48	1.75	0.85
		D	1.86	2.05	2.32	2.53	2.81	3.11	3.52	3.84	1.75	0.85
	Unfastened	A	0.725	0.904	1.16	1.35	1.10	1.37	1.75	2.05	1.85	0.80
		B	1.44	1.66	1.97	2.21	2.18	2.52	2.99	3.36	1.65	0.90
		C	0.684	0.733	0.802	0.856	1.04	1.11	1.22	1.30	1.65	0.90
		D	1.49	1.73	2.07	2.33	2.26	2.62	3.13	3.52	1.90	0.80
400S162-43	Fastened	A	0.479	0.602	0.777	0.911	-	-	-	-	1.75	0.85
		B	0.935	1.09	1.31	1.47	-	-	-	-	1.65	0.90
		C	0.398	0.457	0.541	0.606	-	-	-	-	1.75	0.85
		D	1.16	1.29	1.47	1.62	-	-	-	-	1.75	0.85
	Unfastened	A	0.479	0.602	0.777	0.911	-	-	-	-	1.85	0.80
		B	0.935	1.09	1.31	1.47	-	-	-	-	1.65	0.90
		C	0.409	0.441	0.487	0.522	-	-	-	-	1.65	0.90
		D	0.944	1.11	1.33	1.51	-	-	-	-	1.90	0.80
400S162-33	Fastened	A	0.285	0.362	0.471	0.555	-	-	-	-	1.75	0.85
		B	0.530	0.625	0.758	0.861	-	-	-	-	1.65	0.90
		C	0.214	0.249	0.298	0.336	-	-	-	-	1.75	0.85
		D	0.650	0.731	0.846	0.934	-	-	-	-	1.75	0.85
	Unfastened	A	0.285	0.362	0.471	0.555	-	-	-	-	1.85	0.80
		B	0.530	0.625	0.758	0.861	-	-	-	-	1.65	0.90
		C	0.199	0.216	0.241	0.260	-	-	-	-	1.65	0.90
		D	0.384	0.455	0.556	0.633	-	-	-	-	1.90	0.80
362S162-68	Fastened	A	1.11	1.37	1.74	2.02	1.68	2.08	2.64	3.07	1.75	0.85
		B	2.23	2.54	2.99	3.34	3.37	3.85	4.53	5.05	1.65	0.90
		C	1.11	1.25	1.45	1.61	1.68	1.90	2.20	2.44	1.75	0.85
		D	3.02	3.31	3.72	4.03	4.58	5.02	5.64	6.11	1.75	0.85
	Unfastened	A	1.11	1.37	1.74	2.02	1.68	2.08	2.64	3.07	1.85	0.80
		B	2.23	2.54	2.99	3.34	3.37	3.85	4.53	5.05	1.65	0.90
		C	1.15	1.22	1.33	1.41	1.74	1.85	2.01	2.14	1.65	0.90
		D	2.27	2.60	3.08	3.45	3.43	3.95	4.67	5.23	1.90	0.80
362S162-54	Fastened	A	0.733	0.913	1.17	1.37	1.11	1.38	1.77	2.07	1.75	0.85
		B	1.45	1.67	1.98	2.22	2.19	2.53	3.01	3.37	1.65	0.90
		C	0.679	0.773	0.907	1.01	1.03	1.17	1.37	1.53	1.75	0.85
		D	1.89	2.09	2.36	2.58	2.86	3.16	3.58	3.91	1.75	0.85
	Unfastened	A	0.733	0.913	1.17	1.37	1.11	1.38	1.77	2.07	1.85	0.80
		B	1.45	1.67	1.98	2.22	2.19	2.53	3.01	3.37	1.65	0.90
		C	0.700	0.751	0.822	0.877	1.06	1.14	1.25	1.33	1.65	0.90
		D	1.49	1.73	2.07	2.33	2.26	2.62	3.13	3.52	1.90	0.80

Table II - 14

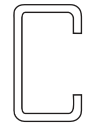
Web Crippling, P_n, kips [1,2,4]
SSMA Studs
C-Sections With Lips



Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi, Bearing Length, N, in.[4]				F_y = 50 ksi, Bearing Length, N, in.[4]				Ω_w	ϕ_w
			1	2	4	6	1	2	4	6		
362S162-43	Fastened	A	0.484	0.609	0.786	0.921	-	-	-	-	1.75	0.85
		B	0.940	1.09	1.31	1.48	-	-	-	-	1.65	0.90
		C	0.414	0.476	0.563	0.630	-	-	-	-	1.75	0.85
		D	1.18	1.32	1.50	1.65	-	-	-	-	1.75	0.85
	Unfastened	A	0.484	0.609	0.786	0.921	-	-	-	-	1.85	0.80
		B	0.940	1.09	1.31	1.48	-	-	-	-	1.65	0.90
		C	0.421	0.454	0.501	0.537	-	-	-	-	1.65	0.90
		D	0.945	1.11	1.34	1.51	-	-	-	-	1.90	0.80
362S162-33	Fastened	A	0.289	0.367	0.478	0.563	-	-	-	-	1.75	0.85
		B	0.534	0.628	0.763	0.866	-	-	-	-	1.65	0.90
		C	0.225	0.262	0.313	0.353	-	-	-	-	1.75	0.85
		D	0.666	0.749	0.867	0.957	-	-	-	-	1.75	0.85
	Unfastened	A	0.289	0.367	0.478	0.563	-	-	-	-	1.85	0.80
		B	0.534	0.628	0.763	0.866	-	-	-	-	1.65	0.90
		C	0.206	0.224	0.250	0.269	-	-	-	-	1.65	0.90
		D	0.385	0.456	0.556	0.633	-	-	-	-	1.90	0.80
350S162-68	Fastened	A	1.11	1.38	1.75	2.03	1.69	2.08	2.65	3.08	1.75	0.85
		B	2.23	2.55	3.00	3.34	3.38	3.86	4.54	5.06	1.65	0.90
		C	1.12	1.26	1.47	1.62	1.70	1.92	2.22	2.46	1.75	0.85
		D	3.04	3.33	3.74	4.05	4.60	5.04	5.67	6.14	1.75	0.85
	Unfastened	A	1.11	1.38	1.75	2.03	1.69	2.08	2.65	3.08	1.85	0.80
		B	2.23	2.55	3.00	3.34	3.38	3.86	4.54	5.06	1.65	0.90
		C	1.16	1.23	1.34	1.42	1.75	1.87	2.03	2.15	1.65	0.90
		D	2.27	2.61	3.08	3.45	3.44	3.95	4.67	5.23	1.90	0.80
350S162-54	Fastened	A	0.735	0.917	1.17	1.37	1.11	1.39	1.78	2.08	1.75	0.85
		B	1.45	1.67	1.99	2.23	2.20	2.53	3.01	3.38	1.65	0.90
		C	0.687	0.782	0.917	1.02	1.04	1.18	1.39	1.55	1.75	0.85
		D	1.90	2.10	2.38	2.59	2.88	3.18	3.60	3.93	1.75	0.85
	Unfastened	A	0.735	0.917	1.17	1.37	1.11	1.39	1.78	2.08	1.85	0.80
		B	1.45	1.67	1.99	2.23	2.20	2.53	3.01	3.38	1.65	0.90
		C	0.706	0.757	0.829	0.884	1.07	1.15	1.26	1.34	1.65	0.90
		D	1.49	1.73	2.07	2.33	2.26	2.62	3.13	3.53	1.90	0.80
350S162-43	Fastened	A	0.486	0.612	0.789	0.925	-	-	-	-	1.75	0.85
		B	0.942	1.10	1.32	1.48	-	-	-	-	1.65	0.90
		C	0.419	0.482	0.570	0.638	-	-	-	-	1.75	0.85
		D	1.19	1.32	1.52	1.66	-	-	-	-	1.75	0.85
	Unfastened	A	0.486	0.612	0.789	0.925	-	-	-	-	1.85	0.80
		B	0.942	1.10	1.32	1.48	-	-	-	-	1.65	0.90
		C	0.425	0.459	0.506	0.543	-	-	-	-	1.65	0.90
		D	0.945	1.11	1.34	1.51	-	-	-	-	1.90	0.80
350S162-33	Fastened	A	0.290	0.369	0.480	0.565	-	-	-	-	1.75	0.85
		B	0.535	0.630	0.764	0.867	-	-	-	-	1.65	0.90
		C	0.229	0.266	0.319	0.359	-	-	-	-	1.75	0.85
		D	0.672	0.755	0.874	0.964	-	-	-	-	1.75	0.85
	Unfastened	A	0.290	0.369	0.480	0.565	-	-	-	-	1.85	0.80
		B	0.535	0.630	0.764	0.867	-	-	-	-	1.65	0.90
		C	0.209	0.227	0.253	0.273	-	-	-	-	1.65	0.90
		D	0.385	0.456	0.556	0.634	-	-	-	-	1.90	0.80

Table II - 14

Web Crippling, P_n, kips[1,2,4]
SSMA Studs
C-Sections With Lips



Section	Fastened or Unfastened	Case[3]	F_y = 33 ksi, Bearing Length, N, in.[4]				F_y = 50 ksi, Bearing Length, N, in.[4]				Ω_w	ϕ_w
			1	2	4	6	1	2	4	6		
250S162-68	Fastened	A	1.14	1.41	1.79	2.08	1.73	2.14	2.72	3.16	1.75	0.85
		B	2.26	2.58	3.03	3.38	3.42	3.91	4.60	5.12	1.65	0.90
		C	1.21	1.37	1.59	1.76	1.84	2.07	2.41	2.66	1.75	0.85
		D	3.18	3.48	3.91	4.24	4.81	5.27	5.92	6.42	1.75	0.85
	Unfastened	A	1.14	1.41	1.79	2.08	1.73	2.14	2.72	3.16	1.85	0.80
		B	2.26	2.58	3.03	3.38	3.42	3.91	4.60	5.12	1.65	0.90
		C	1.23	1.31	1.42	1.51	1.86	1.98	2.16	2.29	1.65	0.90
		D	2.27	2.61	3.09	3.45	3.44	3.95	4.68	5.23	1.90	0.80
250S162-54	Fastened	A	0.757	0.944	1.21	1.41	1.15	1.43	1.83	2.14	1.75	0.85
		B	1.47	1.70	2.01	2.26	2.23	2.57	3.05	3.42	1.65	0.90
		C	0.753	0.857	1.01	1.12	1.14	1.30	1.52	1.69	1.75	0.85
		D	2.00	2.21	2.50	2.73	3.03	3.34	3.79	4.13	1.75	0.85
	Unfastened	A	0.757	0.944	1.21	1.41	1.15	1.43	1.83	2.14	1.85	0.80
		B	1.47	1.70	2.01	2.26	2.23	2.57	3.05	3.42	1.65	0.90
		C	0.758	0.813	0.890	0.949	1.15	1.23	1.35	1.44	1.65	0.90
		D	1.49	1.73	2.07	2.33	2.26	2.62	3.14	3.53	1.90	0.80
250S162-43	Fastened	A	0.503	0.632	0.816	0.956	-	-	-	-	1.75	0.85
		B	0.956	1.11	1.34	1.51	-	-	-	-	1.65	0.90
		C	0.467	0.537	0.636	0.712	-	-	-	-	1.75	0.85
		D	1.26	1.40	1.61	1.76	-	-	-	-	1.75	0.85
	Unfastened	A	0.503	0.632	0.816	0.956	-	-	-	-	1.85	0.80
		B	0.956	1.11	1.34	1.51	-	-	-	-	1.65	0.90
		C	0.462	0.498	0.550	0.589	-	-	-	-	1.65	0.90
		D	0.946	1.11	1.34	1.51	-	-	-	-	1.90	0.80
250S162-33	Fastened	A	0.302	0.384	0.499	0.588	-	-	-	-	1.75	0.85
		B	0.544	0.641	0.778	0.883	-	-	-	-	1.65	0.90
		C	0.262	0.305	0.365	0.411	-	-	-	-	1.75	0.85
		D	0.720	0.810	0.937	1.03	-	-	-	-	1.75	0.85
	Unfastened	A	0.302	0.384	0.499	0.588	-	-	-	-	1.85	0.80
		B	0.544	0.641	0.778	0.883	-	-	-	-	1.65	0.90
		C	0.231	0.251	0.280	0.302	-	-	-	-	1.65	0.90
		D	0.385	0.457	0.557	0.635	-	-	-	-	1.90	0.80

Notes:

1. Web crippling strengths are nominal strengths calculated without consideration of holes or other openings. To obtain available strengths, the values must be modified by safety factors (ASD) or resistance factors (LRFD), which are provided in the table.
2. Strength reductions factors for openings must be calculated in accordance with the provisions of *Specification* Section C3.4.2.
3. Case A
   End Reaction, Opposing Loads Spaced > 1.5h

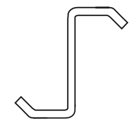
   Case B
   Interior Reactions, Opposing Loads Spaced > 1.5h

   Case C
   End Reaction, Opposing Loads Spaced ≤ 1.5h

   Case D
   Interior Reactions, Opposing Loads Spaced ≤ 1.5h
4. Linear interpolation is permitted between bearing lengths.

Table II - 15

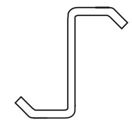
Web Crippling, P_n, kips[1,3]
Z-Sections With Lips



Section	Fastened or Unfastened	Case[2]	F_y = 33 ksi, Bearing Length, N, in.[3]				F_y = 55 ksi, Bearing Length, N, in.[3]				Ω_w	ϕ_w
			1	2	4	6	1	2	4	6		
12ZS3.25x105	Fastened	A	1.95	2.37	2.96	3.41	3.25	3.94	4.93	5.69	1.75	0.85
		B	4.20	4.73	5.47	6.04	7.00	7.88	9.12	10.1	1.65	0.90
		C	2.09	2.38	2.78	3.09	3.48	3.96	4.63	5.15	1.75	0.85
		D	5.61	6.02	6.61	7.06	9.35	10.0	11.0	11.8	1.85	0.80
	Unfastened	A	1.68	1.72	1.78	1.82	2.80	2.87	2.97	3.04	1.80	0.85
		B	4.20	4.73	5.47	6.04	7.00	7.88	9.12	10.1	1.65	0.90
		C	1.82	1.92	2.07	2.17	3.04	3.20	3.44	3.62	1.65	0.90
		D	3.86	4.36	5.08	5.63	6.43	7.27	8.46	9.38	1.90	0.80
12ZS3.25x085	Fastened	A	1.28	1.56	1.97	2.29	2.13	2.61	3.29	3.81	1.75	0.85
		B	2.67	3.03	3.54	3.93	4.45	5.05	5.89	6.54	1.65	0.90
		C	1.22	1.40	1.65	1.85	2.03	2.33	2.75	3.08	1.75	0.85
		D	3.41	3.68	4.07	4.36	5.68	6.13	6.78	7.27	1.85	0.80
	Unfastened	A	1.09	1.12	1.16	1.19	1.82	1.87	1.93	1.99	1.80	0.85
		B	2.67	3.03	3.54	3.93	4.45	5.05	5.89	6.54	1.65	0.90
		C	1.02	1.08	1.17	1.24	1.70	1.80	1.95	2.06	1.65	0.90
		D	1.95	2.22	2.61	2.91	3.25	3.71	4.35	4.85	1.90	0.80
12ZS3.25x070	Fastened	A	0.861	1.06	1.35	1.57	1.44	1.77	2.25	2.62	1.75	0.85
		B	1.75	2.00	2.35	2.62	2.91	3.33	3.92	4.37	1.65	0.90
		C	0.716	0.828	0.986	1.11	1.19	1.38	1.64	1.85	1.75	0.85
		D	2.12	2.30	2.56	2.76	3.53	3.84	4.27	4.60	1.85	0.80
	Unfastened	A	0.732	0.753	0.783	0.807	1.22	1.26	1.31	1.34	1.80	0.85
		B	1.75	2.00	2.35	2.62	2.91	3.33	3.92	4.37	1.65	0.90
		C	0.580	0.619	0.673	0.714	0.967	1.03	1.12	1.19	1.65	0.90
		D	0.894	1.03	1.22	1.36	1.49	1.71	2.03	2.27	1.90	0.80
12ZS2.25x105	Fastened	A	1.95	2.37	2.96	3.41	3.25	3.94	4.93	5.69	1.75	0.85
		B	4.20	4.73	5.47	6.04	7.00	7.88	9.12	10.1	1.65	0.90
		C	2.09	2.38	2.78	3.09	3.48	3.96	4.63	5.15	1.75	0.85
		D	5.61	6.02	6.61	7.06	9.35	10.0	11.0	11.8	1.85	0.80
	Unfastened	A	1.68	1.72	1.78	1.82	2.80	2.87	2.97	3.04	1.80	0.85
		B	4.20	4.73	5.47	6.04	7.00	7.88	9.12	10.1	1.65	0.90
		C	1.82	1.92	2.07	2.17	3.04	3.20	3.44	3.62	1.65	0.90
		D	3.86	4.36	5.08	5.63	6.43	7.27	8.46	9.38	1.90	0.80
12ZS2.25x085	Fastened	A	1.28	1.56	1.97	2.29	2.13	2.61	3.29	3.81	1.75	0.85
		B	2.67	3.03	3.54	3.93	4.45	5.05	5.89	6.54	1.65	0.90
		C	1.22	1.40	1.65	1.85	2.03	2.33	2.75	3.08	1.75	0.85
		D	3.41	3.68	4.07	4.36	5.68	6.13	6.78	7.27	1.85	0.80
	Unfastened	A	1.09	1.12	1.16	1.19	1.82	1.87	1.93	1.99	1.80	0.85
		B	2.67	3.03	3.54	3.93	4.45	5.05	5.89	6.54	1.65	0.90
		C	1.02	1.08	1.17	1.24	1.70	1.80	1.95	2.06	1.65	0.90
		D	1.95	2.22	2.61	2.91	3.25	3.71	4.35	4.85	1.90	0.80
12ZS2.25x070	Fastened	A	0.861	1.06	1.35	1.57	1.44	1.77	2.25	2.62	1.75	0.85
		B	1.75	2.00	2.35	2.62	2.91	3.33	3.92	4.37	1.65	0.90
		C	0.716	0.828	0.986	1.11	1.19	1.38	1.64	1.85	1.75	0.85
		D	2.12	2.30	2.56	2.76	3.53	3.84	4.27	4.60	1.85	0.80
	Unfastened	A	0.732	0.753	0.783	0.807	1.22	1.26	1.31	1.34	1.80	0.85
		B	1.75	2.00	2.35	2.62	2.91	3.33	3.92	4.37	1.65	0.90
		C	0.580	0.619	0.673	0.714	0.967	1.03	1.12	1.19	1.65	0.90
		D	0.894	1.03	1.22	1.36	1.49	1.71	2.03	2.27	1.90	0.80

Table II - 15

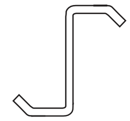
Web Crippling, P_n, kips [1,3]
Z-Sections With Lips



Section	Fastened or Unfastened	Case[2]	F_y = 33 ksi, Bearing Length, N, in.[3] 1	2	4	6	F_y = 55 ksi, Bearing Length, N, in.[3] 1	2	4	6	Ω_w	ϕ_w
10ZS2.75x105	Fastened	A	1.99	2.42	3.03	3.50	3.32	4.04	5.05	5.83	1.75	0.85
		B	4.25	4.78	5.53	6.10	7.08	7.96	9.21	10.2	1.65	0.90
		C	2.32	2.63	3.08	3.43	3.86	4.39	5.14	5.71	1.75	0.85
		D	5.98	6.42	7.04	7.52	9.97	10.7	11.7	12.5	1.85	0.80
	Unfastened	A	1.68	1.72	1.78	1.82	2.81	2.87	2.97	3.04	1.80	0.85
		B	4.25	4.78	5.53	6.10	7.08	7.96	9.21	10.2	1.65	0.90
		C	1.94	2.05	2.20	2.32	3.24	3.41	3.67	3.86	1.65	0.90
		D	3.86	4.37	5.08	5.63	6.43	7.28	8.47	9.39	1.90	0.80
10ZS2.75x085	Fastened	A	1.31	1.61	2.03	2.35	2.19	2.68	3.38	3.91	1.75	0.85
		B	2.70	3.07	3.58	3.97	4.50	5.11	5.96	6.62	1.65	0.90
		C	1.39	1.59	1.88	2.10	2.32	2.66	3.14	3.50	1.75	0.85
		D	3.68	3.97	4.39	4.71	6.13	6.62	7.31	7.85	1.85	0.80
	Unfastened	A	1.09	1.12	1.16	1.19	1.82	1.87	1.94	1.99	1.80	0.85
		B	2.70	3.07	3.58	3.97	4.50	5.11	5.96	6.62	1.65	0.90
		C	1.10	1.17	1.26	1.34	1.84	1.95	2.10	2.23	1.65	0.90
		D	1.95	2.23	2.62	2.91	3.25	3.71	4.36	4.86	1.90	0.80
10ZS2.75x070	Fastened	A	0.889	1.10	1.39	1.62	1.48	1.83	2.32	2.70	1.75	0.85
		B	1.77	2.02	2.38	2.66	2.95	3.37	3.97	4.43	1.65	0.90
		C	0.846	0.978	1.17	1.31	1.41	1.63	1.94	2.18	1.75	0.85
		D	2.32	2.52	2.81	3.03	3.87	4.21	4.68	5.04	1.85	0.80
	Unfastened	A	0.733	0.754	0.784	0.808	1.22	1.26	1.31	1.35	1.80	0.85
		B	1.77	2.02	2.38	2.66	2.95	3.37	3.97	4.43	1.65	0.90
		C	0.636	0.678	0.737	0.783	1.06	1.13	1.23	1.30	1.65	0.90
		D	0.895	1.03	1.22	1.36	1.49	1.72	2.03	2.27	1.90	0.80
10ZS2.75x065	Fastened	A	0.765	0.948	1.21	1.41	1.27	1.58	2.01	2.34	1.75	0.85
		B	1.50	1.73	2.04	2.28	2.51	2.88	3.40	3.80	1.65	0.90
		C	0.694	0.805	0.962	1.08	1.16	1.34	1.60	1.80	1.75	0.85
		D	1.94	2.11	2.36	2.55	3.24	3.52	3.93	4.25	1.85	0.80
	Unfastened	A	0.629	0.648	0.675	0.696	1.05	1.08	1.12	1.16	1.80	0.85
		B	1.50	1.73	2.04	2.28	2.51	2.88	3.40	3.80	1.65	0.90
		C	0.511	0.546	0.595	0.633	0.852	0.910	0.992	1.05	1.65	0.90
		D	0.613	0.708	0.841	0.943	1.02	1.18	1.40	1.57	1.90	0.80
10ZS2.75x059	Fastened	A	0.628	0.782	0.999	1.17	1.05	1.30	1.66	1.94	1.75	0.85
		B	1.21	1.40	1.66	1.86	2.02	2.33	2.76	3.09	1.65	0.90
		C	0.531	0.618	0.742	0.837	0.885	1.03	1.24	1.39	1.75	0.85
		D	1.53	1.67	1.87	2.03	2.55	2.79	3.12	3.38	1.85	0.80
	Unfastened	A	0.515	0.532	0.554	0.572	0.859	0.886	0.924	0.954	1.80	0.85
		B	1.21	1.40	1.66	1.86	2.02	2.33	2.76	3.09	1.65	0.90
		C	0.381	0.408	0.446	0.475	0.635	0.679	0.743	0.792	1.65	0.90
		D	0.321	0.372	0.444	0.499	0.536	0.620	0.740	0.832	1.90	0.80
10ZS2.25x105	Fastened	A	1.99	2.42	3.03	3.50	3.32	4.04	5.05	5.83	1.75	0.85
		B	4.25	4.78	5.53	6.10	7.08	7.96	9.21	10.2	1.65	0.90
		C	2.32	2.63	3.08	3.43	3.86	4.39	5.14	5.71	1.75	0.85
		D	5.98	6.42	7.04	7.52	9.97	10.7	11.7	12.5	1.85	0.80
	Unfastened	A	1.68	1.72	1.78	1.82	2.81	2.87	2.97	3.04	1.80	0.85
		B	4.25	4.78	5.53	6.10	7.08	7.96	9.21	10.2	1.65	0.90
		C	1.94	2.05	2.20	2.32	3.24	3.41	3.67	3.86	1.65	0.90
		D	3.86	4.37	5.08	5.63	6.43	7.28	8.47	9.39	1.90	0.80

Table II - 15

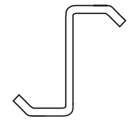
Web Crippling, P_n, kips[1,3]
Z-Sections With Lips



Section	Fastened or Unfas-tened	Case[2]	F_y = 33 ksi				F_y = 55 ksi				Ω_w	ϕ_w
			Bearing Length, N, in.[3]				Bearing Length, N, in.[3]					
			1	2	4	6	1	2	4	6		
10ZS2.25x085	Fastened	A	1.31	1.61	2.03	2.35	2.19	2.68	3.38	3.91	1.75	0.85
		B	2.70	3.07	3.58	3.97	4.50	5.11	5.96	6.62	1.65	0.90
		C	1.39	1.59	1.88	2.10	2.32	2.66	3.14	3.50	1.75	0.85
		D	3.68	3.97	4.39	4.71	6.13	6.62	7.31	7.85	1.85	0.80
	Unfastened	A	1.09	1.12	1.16	1.19	1.82	1.87	1.94	1.99	1.80	0.85
		B	2.70	3.07	3.58	3.97	4.50	5.11	5.96	6.62	1.65	0.90
		C	1.10	1.17	1.26	1.34	1.84	1.95	2.10	2.23	1.65	0.90
		D	1.95	2.23	2.62	2.91	3.25	3.71	4.36	4.86	1.90	0.80
10ZS2.25x070	Fastened	A	0.889	1.10	1.39	1.62	1.48	1.83	2.32	2.70	1.75	0.85
		B	1.77	2.02	2.38	2.66	2.95	3.37	3.97	4.43	1.65	0.90
		C	0.846	0.978	1.17	1.31	1.41	1.63	1.94	2.18	1.75	0.85
		D	2.32	2.52	2.81	3.03	3.87	4.21	4.68	5.04	1.85	0.80
	Unfastened	A	0.733	0.754	0.784	0.808	1.22	1.26	1.31	1.35	1.80	0.85
		B	1.77	2.02	2.38	2.66	2.95	3.37	3.97	4.43	1.65	0.90
		C	0.636	0.678	0.737	0.783	1.06	1.13	1.23	1.30	1.65	0.90
		D	0.895	1.03	1.22	1.36	1.49	1.72	2.03	2.27	1.90	0.80
10ZS2.25x065	Fastened	A	0.765	0.948	1.21	1.41	1.27	1.58	2.01	2.34	1.75	0.85
		B	1.50	1.73	2.04	2.28	2.51	2.88	3.40	3.80	1.65	0.90
		C	0.694	0.805	0.962	1.08	1.16	1.34	1.60	1.80	1.75	0.85
		D	1.94	2.11	2.36	2.55	3.24	3.52	3.93	4.25	1.85	0.80
	Unfastened	A	0.629	0.648	0.675	0.696	1.05	1.08	1.12	1.16	1.80	0.85
		B	1.50	1.73	2.04	2.28	2.51	2.88	3.40	3.80	1.65	0.90
		C	0.511	0.546	0.595	0.633	0.852	0.910	0.992	1.05	1.65	0.90
		D	0.613	0.708	0.841	0.943	1.02	1.18	1.40	1.57	1.90	0.80
10ZS2.25x059	Fastened	A	0.628	0.782	0.999	1.17	1.05	1.30	1.66	1.94	1.75	0.85
		B	1.21	1.40	1.66	1.86	2.02	2.33	2.76	3.09	1.65	0.90
		C	0.531	0.618	0.742	0.837	0.885	1.03	1.24	1.39	1.75	0.85
		D	1.53	1.67	1.87	2.03	2.55	2.79	3.12	3.38	1.85	0.80
	Unfastened	A	0.515	0.532	0.554	0.572	0.859	0.886	0.924	0.954	1.80	0.85
		B	1.21	1.40	1.66	1.86	2.02	2.33	2.76	3.09	1.65	0.90
		C	0.381	0.408	0.446	0.475	0.635	0.679	0.743	0.792	1.65	0.90
		D	0.321	0.372	0.444	0.499	0.536	0.620	0.740	0.832	1.90	0.80
8ZS2.75x105	Fastened	A	2.05	2.49	3.11	3.59	3.41	4.15	5.18	5.98	1.75	0.85
		B	4.30	4.83	5.59	6.18	7.16	8.06	9.32	10.3	1.65	0.90
		C	2.57	2.92	3.42	3.80	4.28	4.87	5.70	6.34	1.75	0.85
		D	6.39	6.86	7.52	8.03	10.6	11.4	12.5	13.4	1.85	0.80
	Unfastened	A	1.68	1.73	1.78	1.83	2.81	2.88	2.97	3.04	1.80	0.85
		B	4.30	4.83	5.59	6.18	7.16	8.06	9.32	10.3	1.65	0.90
		C	2.07	2.19	2.35	2.48	3.46	3.65	3.92	4.13	1.65	0.90
		D	3.86	4.37	5.09	5.64	6.44	7.29	8.48	9.40	1.90	0.80
8ZS2.75x085	Fastened	A	1.35	1.66	2.09	2.42	2.25	2.76	3.48	4.03	1.75	0.85
		B	2.74	3.11	3.63	4.03	4.56	5.18	6.04	6.71	1.65	0.90
		C	1.58	1.81	2.14	2.39	2.63	3.02	3.56	3.98	1.75	0.85
		D	3.98	4.30	4.75	5.09	6.63	7.16	7.91	8.49	1.85	0.80
	Unfastened	A	1.09	1.12	1.16	1.20	1.82	1.87	1.94	1.99	1.80	0.85
		B	2.74	3.11	3.63	4.03	4.56	5.18	6.04	6.71	1.65	0.90
		C	1.19	1.26	1.37	1.44	1.99	2.11	2.28	2.41	1.65	0.90
		D	1.95	2.23	2.62	2.92	3.26	3.72	4.36	4.86	1.90	0.80

Table II - 15

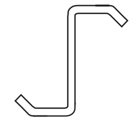
Web Crippling, P_n, kips [1,3]
Z-Sections With Lips



Section	Fastened or Unfas-tened	Case[2]	F_y = 33 ksi				F_y = 55 ksi				Ω_w	ϕ_w
			Bearing Length, N, in.[3]				Bearing Length, N, in.[3]					
			1	2	4	6	1	2	4	6		
8ZS2.75x070	Fastened	A	0.919	1.14	1.44	1.68	1.53	1.89	2.40	2.79	1.75	0.85
		B	1.80	2.05	2.42	2.70	3.00	3.42	4.03	4.50	1.65	0.90
		C	0.991	1.15	1.36	1.53	1.65	1.91	2.27	2.55	1.75	0.85
		D	2.55	2.77	3.08	3.32	4.25	4.61	5.14	5.53	1.85	0.80
	Unfastened	A	0.734	0.755	0.785	0.809	1.22	1.26	1.31	1.35	1.80	0.85
		B	1.80	2.05	2.42	2.70	3.00	3.42	4.03	4.50	1.65	0.90
		C	0.698	0.744	0.809	0.859	1.16	1.24	1.35	1.43	1.65	0.90
		D	0.896	1.03	1.22	1.37	1.49	1.72	2.03	2.28	1.90	0.80
8ZS2.75x065	Fastened	A	0.792	0.982	1.25	1.46	1.32	1.64	2.08	2.43	1.75	0.85
		B	1.53	1.75	2.07	2.31	2.55	2.92	3.45	3.85	1.65	0.90
		C	0.825	0.957	1.14	1.29	1.38	1.60	1.91	2.14	1.75	0.85
		D	2.14	2.34	2.61	2.81	3.57	3.89	4.34	4.69	1.85	0.80
	Unfastened	A	0.630	0.649	0.676	0.696	1.05	1.08	1.13	1.16	1.80	0.85
		B	1.53	1.75	2.07	2.31	2.55	2.92	3.45	3.85	1.65	0.90
		C	0.565	0.603	0.657	0.699	0.941	1.00	1.10	1.16	1.65	0.90
		D	0.614	0.708	0.842	0.944	1.02	1.18	1.40	1.57	1.90	0.80
8ZS2.75x059	Fastened	A	0.652	0.811	1.04	1.21	1.09	1.35	1.73	2.02	1.75	0.85
		B	1.23	1.42	1.68	1.89	2.05	2.36	2.80	3.14	1.65	0.90
		C	0.646	0.752	0.902	1.02	1.08	1.25	1.50	1.70	1.75	0.85
		D	1.71	1.86	2.09	2.26	2.84	3.11	3.48	3.76	1.85	0.80
	Unfastened	A	0.516	0.532	0.555	0.573	0.860	0.887	0.925	0.955	1.80	0.85
		B	1.23	1.42	1.68	1.89	2.05	2.36	2.80	3.14	1.65	0.90
		C	0.424	0.455	0.497	0.530	0.707	0.758	0.828	0.883	1.65	0.90
		D	0.322	0.373	0.445	0.500	0.536	0.621	0.741	0.833	1.90	0.80
8ZS2.25x105	Fastened	A	2.05	2.49	3.11	3.59	3.41	4.15	5.18	5.98	1.75	0.85
		B	4.30	4.83	5.59	6.18	7.16	8.06	9.32	10.3	1.65	0.90
		C	2.57	2.92	3.42	3.80	4.28	4.87	5.70	6.34	1.75	0.85
		D	6.39	6.86	7.52	8.03	10.6	11.4	12.5	13.4	1.85	0.80
	Unfastened	A	1.68	1.73	1.78	1.83	2.81	2.88	2.97	3.04	1.80	0.85
		B	4.30	4.83	5.59	6.18	7.16	8.06	9.32	10.3	1.65	0.90
		C	2.07	2.19	2.35	2.48	3.46	3.65	3.92	4.13	1.65	0.90
		D	3.86	4.37	5.09	5.64	6.44	7.29	8.48	9.40	1.90	0.80
8ZS2.25x085	Fastened	A	1.35	1.66	2.09	2.42	2.25	2.76	3.48	4.03	1.75	0.85
		B	2.74	3.11	3.63	4.03	4.56	5.18	6.04	6.71	1.65	0.90
		C	1.58	1.81	2.14	2.39	2.63	3.02	3.56	3.98	1.75	0.85
		D	3.98	4.30	4.75	5.09	6.63	7.16	7.91	8.49	1.85	0.80
	Unfastened	A	1.09	1.12	1.16	1.20	1.82	1.87	1.94	1.99	1.80	0.85
		B	2.74	3.11	3.63	4.03	4.56	5.18	6.04	6.71	1.65	0.90
		C	1.19	1.26	1.37	1.44	1.99	2.11	2.28	2.41	1.65	0.90
		D	1.95	2.23	2.62	2.92	3.26	3.72	4.36	4.86	1.90	0.80
8ZS2.25x070	Fastened	A	0.919	1.14	1.44	1.68	1.53	1.89	2.40	2.79	1.75	0.85
		B	1.80	2.05	2.42	2.70	3.00	3.42	4.03	4.50	1.65	0.90
		C	0.991	1.15	1.36	1.53	1.65	1.91	2.27	2.55	1.75	0.85
		D	2.55	2.77	3.08	3.32	4.25	4.61	5.14	5.53	1.85	0.80
	Unfastened	A	0.734	0.755	0.785	0.809	1.22	1.26	1.31	1.35	1.80	0.85
		B	1.80	2.05	2.42	2.70	3.00	3.42	4.03	4.50	1.65	0.90
		C	0.698	0.744	0.809	0.859	1.16	1.24	1.35	1.43	1.65	0.90
		D	0.896	1.03	1.22	1.37	1.49	1.72	2.03	2.28	1.90	0.80

Table II - 15

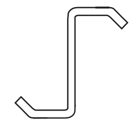
Web Crippling, P_n, kips [1,3]
Z-Sections With Lips



Section	Fastened or Unfastened	Case[2]	F_y = 33 ksi				F_y = 55 ksi				Ω_w	ϕ_w
			Bearing Length, N, in.[3]				Bearing Length, N, in.[3]					
			1	2	4	6	1	2	4	6		
8ZS2.25x065	Fastened	A	0.792	0.982	1.25	1.46	1.32	1.64	2.08	2.43	1.75	0.85
		B	1.53	1.75	2.07	2.31	2.55	2.92	3.45	3.85	1.65	0.90
		C	0.825	0.957	1.14	1.29	1.38	1.60	1.91	2.14	1.75	0.85
		D	2.14	2.34	2.61	2.81	3.57	3.89	4.34	4.69	1.85	0.80
	Unfastened	A	0.630	0.649	0.676	0.696	1.05	1.08	1.13	1.16	1.80	0.85
		B	1.53	1.75	2.07	2.31	2.55	2.92	3.45	3.85	1.65	0.90
		C	0.565	0.603	0.657	0.699	0.941	1.00	1.10	1.16	1.65	0.90
		D	0.614	0.708	0.842	0.944	1.02	1.18	1.40	1.57	1.90	0.80
8ZS2.25x059	Fastened	A	0.652	0.811	1.04	1.21	1.09	1.35	1.73	2.02	1.75	0.85
		B	1.23	1.42	1.68	1.89	2.05	2.36	2.80	3.14	1.65	0.90
		C	0.646	0.752	0.902	1.02	1.08	1.25	1.50	1.70	1.75	0.85
		D	1.71	1.86	2.09	2.26	2.84	3.11	3.48	3.76	1.85	0.80
	Unfastened	A	0.516	0.532	0.555	0.573	0.860	0.887	0.925	0.955	1.80	0.85
		B	1.23	1.42	1.68	1.89	2.05	2.36	2.80	3.14	1.65	0.90
		C	0.424	0.455	0.497	0.530	0.707	0.758	0.828	0.883	1.65	0.90
		D	0.322	0.373	0.445	0.500	0.536	0.621	0.741	0.833	1.90	0.80
6ZS2.25x105	Fastened	A	2.11	2.56	3.20	3.69	3.51	4.27	5.34	6.16	1.75	0.85
		B	4.35	4.90	5.67	6.26	7.26	8.16	9.45	10.4	1.65	0.90
		C	2.86	3.25	3.81	4.23	4.77	5.42	6.34	7.05	1.75	0.85
		D	6.86	7.36	8.08	8.63	11.4	12.3	13.5	14.4	1.85	0.80
	Unfastened	A	1.69	1.73	1.78	1.83	2.81	2.88	2.97	3.05	1.80	0.85
		B	4.35	4.90	5.67	6.26	7.26	8.16	9.45	10.4	1.65	0.90
		C	2.23	2.35	2.53	2.66	3.71	3.92	4.21	4.43	1.65	0.90
		D	3.87	4.38	5.09	5.64	6.45	7.29	8.49	9.41	1.90	0.80
6ZS2.25x085	Fastened	A	1.40	1.71	2.16	2.50	2.33	2.85	3.60	4.17	1.75	0.85
		B	2.78	3.15	3.68	4.09	4.63	5.25	6.13	6.81	1.65	0.90
		C	1.79	2.06	2.43	2.72	2.99	3.43	4.05	4.53	1.75	0.85
		D	4.32	4.67	5.16	5.53	7.20	7.78	8.60	9.22	1.85	0.80
	Unfastened	A	1.09	1.12	1.17	1.20	1.82	1.87	1.94	1.99	1.80	0.85
		B	2.78	3.15	3.68	4.09	4.63	5.25	6.13	6.81	1.65	0.90
		C	1.29	1.37	1.48	1.57	2.16	2.29	2.47	2.62	1.65	0.90
		D	1.96	2.23	2.62	2.92	3.26	3.72	4.37	4.87	1.90	0.80
6ZS2.25x070	Fastened	A	0.953	1.18	1.50	1.74	1.59	1.96	2.49	2.90	1.75	0.85
		B	1.83	2.09	2.46	2.74	3.05	3.48	4.10	4.57	1.65	0.90
		C	1.16	1.34	1.59	1.79	1.93	2.23	2.66	2.98	1.75	0.85
		D	2.81	3.05	3.39	3.66	4.68	5.08	5.66	6.10	1.85	0.80
	Unfastened	A	0.735	0.756	0.787	0.810	1.22	1.26	1.31	1.35	1.80	0.85
		B	1.83	2.09	2.46	2.74	3.05	3.48	4.10	4.57	1.65	0.90
		C	0.769	0.820	0.891	0.946	1.28	1.37	1.49	1.58	1.65	0.90
		D	0.898	1.03	1.22	1.37	1.50	1.72	2.04	2.28	1.90	0.80
6ZS2.25x065	Fastened	A	0.823	1.02	1.30	1.51	1.37	1.70	2.17	2.52	1.75	0.85
		B	1.55	1.78	2.10	2.35	2.59	2.97	3.51	3.92	1.65	0.90
		C	0.975	1.13	1.35	1.52	1.63	1.89	2.25	2.53	1.75	0.85
		D	2.38	2.59	2.89	3.12	3.96	4.31	4.81	5.20	1.85	0.80
	Unfastened	A	0.631	0.650	0.677	0.698	1.05	1.08	1.13	1.16	1.80	0.85
		B	1.55	1.78	2.10	2.35	2.59	2.97	3.51	3.92	1.65	0.90
		C	0.626	0.668	0.728	0.774	1.04	1.11	1.21	1.29	1.65	0.90
		D	0.615	0.710	0.843	0.946	1.03	1.18	1.41	1.58	1.90	0.80

Table II - 15

Web Crippling, P_n, kips [1,3]
Z-Sections With Lips



Section	Fastened or Unfastened	Case[2]	F_y = 33 ksi Bearing Length, N, in.[3]				F_y = 55 ksi Bearing Length, N, in.[3]				Ω_w	ϕ_w
			1	2	4	6	1	2	4	6		
6ZS2.25x059	Fastened	A	0.679	0.845	1.08	1.26	1.13	1.41	1.80	2.10	1.75	0.85
		B	1.25	1.44	1.71	1.92	2.09	2.41	2.86	3.20	1.65	0.90
		C	0.777	0.905	1.09	1.22	1.30	1.51	1.81	2.04	1.75	0.85
		D	1.91	2.08	2.33	2.53	3.18	3.47	3.89	4.21	1.85	0.80
	Unfastened	A	0.517	0.533	0.556	0.574	0.861	0.889	0.927	0.956	1.80	0.85
		B	1.25	1.44	1.71	1.92	2.09	2.41	2.86	3.20	1.65	0.90
		C	0.475	0.508	0.556	0.592	0.791	0.847	0.926	0.987	1.65	0.90
		D	0.322	0.373	0.446	0.501	0.537	0.622	0.743	0.835	1.90	0.80
4ZS2.25x070	Fastened	A	0.995	1.23	1.56	1.82	1.66	2.05	2.60	3.03	1.75	0.85
		B	1.86	2.13	2.51	2.80	3.11	3.55	4.18	4.66	1.65	0.90
		C	1.36	1.57	1.87	2.10	2.26	2.62	3.12	3.50	1.75	0.85
		D	3.12	3.39	3.77	4.06	5.20	5.65	6.29	6.77	1.85	0.80
	Unfastened	A	0.736	0.758	0.788	0.811	1.23	1.26	1.31	1.35	1.80	0.85
		B	1.86	2.13	2.51	2.80	3.11	3.55	4.18	4.66	1.65	0.90
		C	0.854	0.911	0.990	1.05	1.42	1.52	1.65	1.75	1.65	0.90
		D	0.899	1.03	1.22	1.37	1.50	1.72	2.04	2.29	1.90	0.80
4ZS2.25x065	Fastened	A	0.861	1.07	1.36	1.58	1.43	1.78	2.26	2.64	1.75	0.85
		B	1.59	1.82	2.15	2.40	2.64	3.03	3.58	4.00	1.65	0.90
		C	1.16	1.34	1.60	1.80	1.93	2.23	2.67	3.00	1.75	0.85
		D	2.66	2.89	3.23	3.48	4.43	4.82	5.38	5.81	1.85	0.80
	Unfastened	A	0.632	0.651	0.678	0.699	1.05	1.09	1.13	1.16	1.80	0.85
		B	1.59	1.82	2.15	2.40	2.64	3.03	3.58	4.00	1.65	0.90
		C	0.699	0.747	0.814	0.866	1.17	1.24	1.36	1.44	1.65	0.90
		D	0.616	0.711	0.845	0.947	1.03	1.18	1.41	1.58	1.90	0.80
4ZS2.25x059	Fastened	A	0.712	0.886	1.13	1.32	1.19	1.48	1.89	2.20	1.75	0.85
		B	1.28	1.48	1.75	1.96	2.14	2.46	2.92	3.27	1.65	0.90
		C	0.936	1.09	1.31	1.47	1.56	1.82	2.18	2.46	1.75	0.85
		D	2.15	2.35	2.63	2.85	3.58	3.91	4.38	4.74	1.85	0.80
	Unfastened	A	0.518	0.534	0.557	0.575	0.863	0.890	0.929	0.958	1.80	0.85
		B	1.28	1.48	1.75	1.96	2.14	2.46	2.92	3.27	1.65	0.90
		C	0.535	0.573	0.626	0.667	0.892	0.955	1.04	1.11	1.65	0.90
		D	0.323	0.374	0.446	0.502	0.538	0.624	0.744	0.836	1.90	0.80

Notes:

1. Web crippling strengths are nominal strengths calculated without consideration of holes or other openings. To obtain available strengths, the values must be modified by safety factors (ASD) or resistance factors (LRFD), which are provided in the table.
2. Case A

   End Reaction, Opposing Loads Spaced > 1.5h

   Case B

   Interior Reactions, Opposing Loads Spaced > 1.5h

   Case C

   End Reaction, Opposing Loads Spaced ≤ 1.5h

   Case D

   Interior Reactions, Opposing Loads Spaced ≤ 1.5h
3. Linear interpolation is permitted between bearing lengths.

Table II - 16a

Web Crippling Reduction Factor, R_c, for Interior Loading
SSMA Studs
C-Sections with Lips

Loading Condition	Stud Depth (in.)	Hole Depth (in.)	Distance between edge of Hole and Edge of Bearing (in.)						
			0.5	1	2	4	8	12	18
Interior One-Flange N ≥ 3 in.	12	1.5	0.90	0.90	0.90	0.91	0.93	0.95	0.98
	10	1.5	0.90	0.90	0.90	0.91	0.94	0.96	0.99
	8	1.5	0.89	0.90	0.90	0.92	0.95	0.97	1.00
	6	1.5	0.89	0.90	0.91	0.92	0.96	1.00	1.00
	5.5	1.5	0.89	0.90	0.91	0.93	0.97	1.00	1.00
	4	1.5	0.89	0.90	0.91	0.94	0.99	1.00	1.00
	3.625	1.5	0.89	0.89	0.91	0.94	1.00	1.00	1.00
	3.5	1.5	0.89	0.89	0.91	0.94	1.00	1.00	1.00
	2.5	0.75	0.90	0.91	0.93	0.98	1.00	1.00	1.00

Notes:
1. These factors only apply to openings with the listed dimensions.
2. Linear interpolation of R_c is permitted.

Table II - 16b

Web Crippling Reduction Factor, R_c, for End Loading
SSMA Studs
C-Sections with Lips

Loading Condition	Stud Depth (in.)	Hole Depth (in.)	Distance between edge of Hole and Edge of Bearing (in.)						
			0.5	1	1.5	2	3	4	5
End One-Flange N ≥ 1 in.	12	1.5	0.97	0.98	0.98	0.98	0.99	1.00	1.00
	10	1.5	0.96	0.97	0.97	0.98	0.99	0.99	1.00
	8	1.5	0.95	0.96	0.96	0.97	0.98	0.99	1.00
	6	1.5	0.93	0.94	0.95	0.95	0.97	0.98	1.00
	5.5	1.5	0.93	0.93	0.94	0.95	0.96	0.98	1.00
	4	1.5	0.89	0.90	0.91	0.92	0.95	0.97	0.99
	3.625	1.5	0.88	0.89	0.90	0.92	0.94	0.96	0.99
	3.5	1.5	0.87	0.89	0.90	0.91	0.94	0.96	0.99
	2.5	0.75	0.92	0.94	0.96	0.98	1.00	1.00	1.00

Notes:
1. These factors only apply to openings with the listed dimensions.
2. Linear interpolation of R_c is permitted.

SECTION 4 – EXAMPLE PROBLEMS

Example II-1: Four Span Continuous C-Purlins Attached to Through Fastened Roof[1] - LRFD

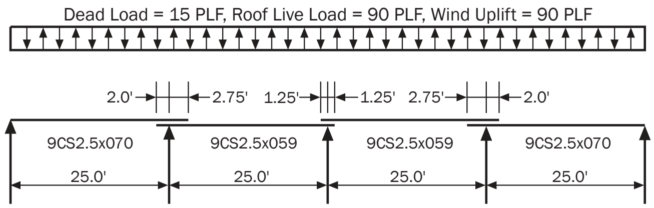


Figure 1 - Spans and Loading
Note: Lap dimensions are to connection points of purlins

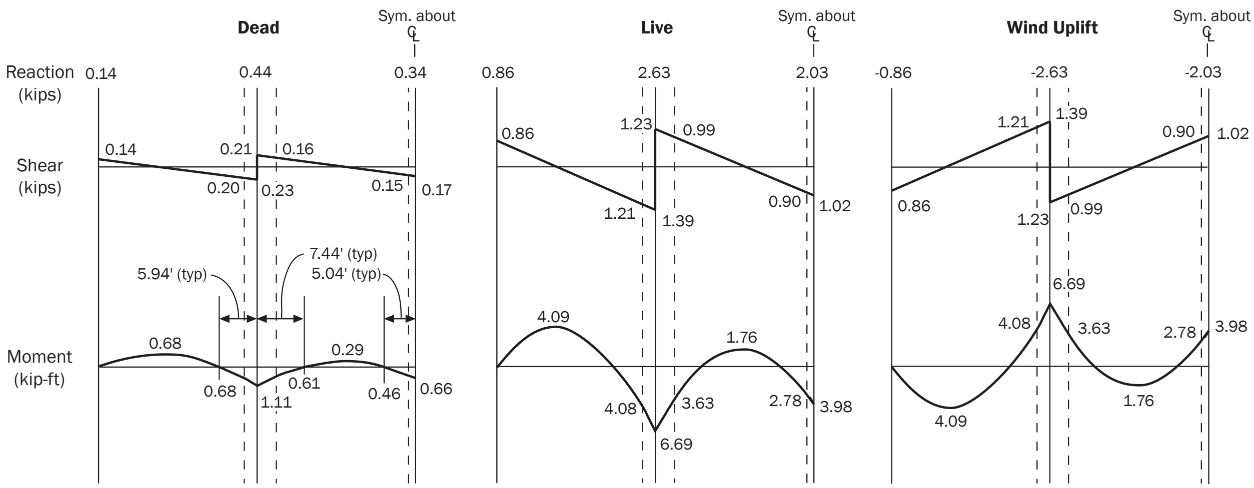


Figure 2 - Reactions, Shears and Moments
Note: Moments and forces are from unfactored nominal loads

Given:

1. Four span C-purlin system using laps at interior support points to create continuity (see Figures 1 and 2)
2. Roof covering is attached with through fasteners along entire length of purlins.
3. F_y = 55 ksi
4. Purlins are lapped back to back over supports but all face in the same direction in a given bay.
5. Bottom flanges are bolted to supporting members with a bearing length of 5 in.
6. The roof covering provides a rotational stiffness to the top flange of the purlins of 0.300 kip-in./rad./in.

Required:

1. Check the design using LRFD with ASCE/SEI 7-05 load combinations for:
   a. Gravity Loads
   b. Uplift Loads

Solution:

---

[1] For design of purlins supporting standing seam roof, see AISI publication *Design Guide for Cold-Formed Steel Purlin Roof Framing Systems.*

1. Assumptions for Analysis and Application of Specification Provisions

The *Specification* does not define the methods of analysis to be used; these judgments are the responsibility of the designer. The following assumptions are considered good practice but are not intended to prohibit other approaches:

a. The purlins are connected within the lapped portions in a manner that achieves full continuity between the individual purlin members.

b. In the continuous beam analysis, the shear and moment diagrams are based on continuous non-prismatic members between supports in which I_x within the lapped portions is the sum of the I_x of the individual members. Gross I_x values are used for the beam analysis.

c. The strength within the lapped portions is assumed to be the sum of the strengths of the individual members.

d. The attachment of the roof covering to the purlin provides continuous lateral-torsional support to the top flange.

e. The roof covering provides continuous distortional bracing of 0.300 kip-in./rad./in.

f. For gravity loads, the region at and near the interior supports is assumed to be not subject to lateral-torsional or distortional buckling between the support and the end of the laps.

g. Under uniform gravity loading, the negative moment region between the end of the lap and the inflection point is assumed to have lateral-torsional and distortional buckling unbraced lengths equal to the distance from the end of the lap to the inflection point.

h. Since the loading, geometry and materials are symmetrical, only the first two spans are checked.

2. Calculation of Section Properties

Based on the design procedures illustrated in Examples I-1 and I-8 and Tables I-1 and II-1 of the AISI *Cold-Formed Steel Design Manual*, the following section properties have been obtained for the two C-sections:

Section	9CS2.5x059	9CS2.5x070
D (in.)	9.00	9.00
t (in.)	0.059	0.070
R (in.)	0.1875	0.1875
A (in.^{2})	0.881	1.05
I_x (in.^{4})	10.3	12.2
S_f (in.^{3})	2.29	2.71
S_e (in.^{3})	1.89	2.47
I_y (in.^{4})	0.698	0.828
r_y (in.)	0.890	0.890
r_o (in.)	3.90	3.90
J (in.^{3})	0.00102	0.00171
C_w (in.^{6})	11.9	14.2

3. Check Gravity Loads

a. Strength for Bending Only (Section C3.1)

Required Strength

Load combinations considered:

1.4D

1.2D + 1.6L_r

By inspection, 1.2D + 1.6L_r controls:

$M_u = 1.2M_D + 1.6M_{Lr}$

End Span, from left to right:

Maximum positive moment:	$M_u = (1.2)(0.68) + (1.6)(4.09) = 7.36$ kip-ft
Negative moment at end of right lap:	$M_u = (1.2)(0.68) + (1.6)(4.08) = 7.34$ kip-ft
Negative moment at support:	$M_u = (1.2)(1.11) + (1.6)(6.69) = 12.0$ kip-ft

Interior span, from left to right:

Negative moment at end of left lap:	$M_u = (1.2)(0.61) + (1.6)(3.63) = 6.54$ kip-ft
Maximum positive moment:	$M_u = (1.2)(0.29) + (1.6)(1.76) = 3.16$ kip-ft
Negative moment at end of right lap:	$M_u = (1.2)(0.46) + (1.6)(2.78) = 5.00$ kip-ft
Negative moment at center support:	$M_u = (1.2)(0.66) + (1.6)(3.98) = 7.16$ kip-ft

Design Flexural Strength

Compute the lowest of the applicable flexural strengths from Sections C3.1.1 (initiation of yielding), C3.1.2 (lateral-torsional buckling), C3.1.4 (distortional buckling) and D6.1.1 (flexural strength of members having tension flange through-fastened to deck or sheathing).

End Span:

At location of maximum positive moment

The section is assumed to be fully braced against lateral-torsional buckling, but distortional buckling and yielding strengths must be calculated.

Calculate the design distortional buckling strength per Section C3.1.4.

A conservative distortional buckling strength can be calculated using Section C3.1.4(a) for members meeting the limitations of that section. In this case, use the more accurate provisions of Section C3.1.4(b) to take advantage of the stiffness provided by the roof covering.

Calculate the elastic distortional buckling stress, F_d.

$$F_d = \beta \frac{k_{\phi fe} + k_{\phi we} + k_\phi}{\tilde{k}_{\phi fg} + \tilde{k}_{\phi wg}} \quad (Eq.\ C3.1.4\text{-}10)$$

The cross section has a single web and a single edge-stiffened flange as required by Section C3.1.4(b). Consider the contribution of the attached roof panel, which has a rotational stiffnesses, k_ϕ = 0.300 kip-in./rad./in. From Table II-7, for the 9CS2.5x070,

$k_{\phi fe} = 0.378$ kips

$\tilde{k}_{\phi fg} = 0.0118$ in.^{2}

$k_{\phi we} = 0.354$ kips

$\tilde{k}_{\phi wg} = 0.00245$ in.^{2}

Since there is no significant moment gradient in the vicinity of the maximum positive moment, use $\beta = 1.0$.

$$F_d = (1.0)\frac{0.378 + 0.354 + 0.300}{0.0118 + 0.00245} = 72.4 \text{ ksi} \qquad (\textit{Eq. } \text{C3.1.4-10})$$

Calculate the design distortional buckling strength per Section C3.1.4

$$M_y = S_{fy}F_y \qquad (\textit{Eq. } \text{C3.1.4-4})$$

$$= (2.71)(55) = 149 \text{ kip-in.}$$

$$M_{crd} = S_f F_d \qquad (\textit{Eq. } \text{C3.1.4-5})$$

$$= (2.71)(72.4) = 196 \text{ kip-in.}$$

$$\lambda_d = \sqrt{M_y / M_{crd}} \qquad (\textit{Eq. } \text{C3.1.4-3})$$

$$= \sqrt{149/196} = 0.872 > 0.673 \text{ therefore,}$$

$$M_n = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (\textit{Eq. } \text{C3.1.4-2})$$

$$= \left(1 - 0.22(1/0.872)\right)(1/0.872)(149) = 128 \text{ kip-in. or } 10.6 \text{ kip-ft}$$

$$\phi_b M_n = (0.90)(10.6) = 9.54 \text{ kip-ft} > 7.36 \text{ kip-ft OK} \qquad (\textit{Eq. } \text{A5.1.1-1})$$

Calculate strength based on initiation of yielding per Section C3.1.1(a)

For the exterior purlin, $t = 0.070$ in.

$$M_n = S_e F_y = (2.47)(55) = 136 \text{ kip-in.} = 11.3 \text{ kip-ft} \qquad (\textit{Eq. } \text{C3.1.1-1})$$

$$\phi_b M_n = (0.95)(11.3) = 10.7 \text{ kip-ft} > 7.36 \text{ kip-ft OK} \qquad (\textit{Eq. } \text{A5.1.1-1})$$

In the region of negative moment between the lap and the inflection point:

Calculate the design lateral-torsional buckling strength per Section C3.1.2.1(a).

Determine the nominal strength using the distance from the inflection point to the lap as the unbraced length using Section C3.1.2.1(a).

$$F_e = \frac{C_b r_o A}{S_f}\sqrt{\sigma_{ey}\sigma_t} \qquad (\textit{Eq. } \text{C3.1.2.1-4})$$

$$L_y = L_t = 5.94 - 2.00 = 3.94 \text{ ft} = 47.3 \text{ in.}$$

$C_b = 1.67$ (Conservatively assumes linear moment diagram in this region.)

$$\sigma_{ey} = \frac{\pi^2 E}{\left(K_y L_y / r_y\right)^2} \qquad (\textit{Eq. } \text{C3.1.2.1-8})$$

$$= \frac{\pi^2 29500}{\left[(1.0)(47.3)/0.890\right]^2} = 103 \text{ ksi}$$

$$\sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2}\right] \qquad (Eq.\ C3.1.2.1\text{-}9)$$

$$= \frac{1}{1.05(3.90)^2}\left[(11300)(0.00171) + \frac{\pi^2(29500)(14.2)}{\left[(1.0)(47.3)\right]^2}\right] = 117 \text{ ksi}$$

$$F_e = \frac{(1.67)(3.90)(1.05)}{2.71}\sqrt{(103)(117)} = 277 \text{ kip-in.} \qquad (Eq.\ C3.1.2.1\text{-}4)$$

$$2.78F_y = (2.78)(55) = 153 \text{ ksi}$$

Since $F_e > 2.78\ F_y$, the section is not subject to lateral-torsional buckling.

Calculate the design distortional buckling strength per Section C3.1.4(b).

Calculate the elastic distortional buckling stress, F_d, for the negative moment region. Since the compression flange has no sheeting, $k_\phi = 0.0$. Use Section C3.1.4(b), since it provides a less conservative result than Section C3.1.4(a).

From Table II-7,

$L_{cr} = 24.1$ in.

$F_d/\beta = 51.3$ ksi

The bottom flange is not restrained from rotation by panel or other discrete bracing. The unbraced length for distortional buckling, L_m, is taken as the distance between the end of the lap and the inflection point.

$L_m = 47.3$ in. (from above).

$$L = \min(L_{cr},\ L_m)$$

$$= \min(24.1,\ 47.3) = 24.1 \text{ in.}$$

The moments at the ends of the segment are:

$M_1 = 0.0$ kip-ft at the inflection point

$M_2 = 7.34$ kip-ft at the lap

$$\beta = 1.0 \le 1 + 0.4(L/L_m)^{0.7}(1 - M_1/M_2)^{0.7} \le 1.3 \qquad (Eq.\ C3.1.4\text{-}11)$$

$$= 1.0 \le 1 + 0.4(24.1/47.3)^{0.7}(1 - 0.0/7.34)^{0.7} \le 1.3$$

$= 1.0 \le 1.25 \le 1.3$ therefore, use $\beta = 1.25$

$F_d = \beta(F_d/\beta)$ (using F_d/β from Table II-7)

$$= 1.25(51.3) = 64.1 \text{ ksi}$$

Calculate the design distortional buckling strength per Section C3.1.4

$$M_y = S_{fy}F_y \qquad (Eq.\ C3.1.4\text{-}4)$$

$$= (2.71)(55) = 149 \text{ kip-in.}$$

$$M_{crd} = S_f F_d \qquad (Eq.\ C3.1.4\text{-}5)$$

$$= (2.71)(64.1) = 174 \text{ kip-in.}$$

$$\lambda_d = \sqrt{M_y / M_{crd}} \qquad (Eq.\ C3.1.4\text{-}3)$$

$$= \sqrt{149/174} = 0.925 > 0.673 \text{ therefore,}$$

$$M_n = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ C3.1.4\text{-}2)$$

$$= (1 - 0.22(1/0.925))(1/0.925)(149) = 123 \text{ kip-in. or } 10.2 \text{ kip-ft}$$

$$\phi_b M_n = (0.90)(10.2) = 9.18 \text{ kip-ft} > 7.34 \text{ kip-ft OK} \qquad (Eq.\ A5.1.1\text{-}1)$$

Calculate the design strength based on the initiation of yielding using Section C3.1.1(a).

$$M_n = S_e F_y = (2.47)(55) = 136 \text{ kip-in. or } 11.3 \text{ kip-ft} \qquad (Eq.\ C3.1.1\text{-}1)$$

$$\phi_b M_n = (0.95)(11.3) = 10.7 \text{ kip-ft} > 7.34 \text{ kip-ft OK} \qquad (Eq.\ A5.1.1\text{-}1)$$

In the lapped region over the support:

The lapped section is assumed to be sufficiently restrained against lateral-torsional buckling and distortional buckling.

Calculate the design flexural strength based on initiation of yielding per Section C3.1.1(a), summing the strength of the two overlapped purlins:

For the interior purlin, t = 0.059 in.

$$M_n = S_e F_y = (1.89)(55) = 104 \text{ kip-in. or } 8.66 \text{ kip-ft} \qquad (Eq.\ C3.1.1\text{-}1)$$

Combined strength of purlins

$$\phi_b M_n = (0.95)(11.3 + 8.66) = 19.0 \text{ kip-ft} > 12.0 \text{ kip-ft OK} \qquad (Eq.\ A5.1.1\text{-}1)$$

Interior Span:

In the region of negative moment between the left lap and the inflection point:

Calculate the design lateral-torsional buckling strength per Section C3.1.2.1(a).

Determine the nominal strength using the distance from the inflection point to the lap as the unbraced length using Section C3.1.2.1(a).

$$L_y = L_t = 7.44 - 2.75 = 4.69 \text{ ft or } 56.3 \text{ in.}$$

C_b = 1.67 (Conservatively assumes linear moment diagram in this region.)

$$\sigma_{ey} = \frac{\pi^2 29500}{\left[(1.0)(56.3)/0.890\right]^2} = 72.8 \text{ ksi} \qquad (Eq.\ C3.1.2.1\text{-}8)$$

$$\sigma_t = \frac{1}{0.881(3.90)^2}\left[(11300)(0.00102) + \frac{\pi^2(29500)(11.9)}{\left[(1.0)(56.3)\right]^2}\right] = 82.4 \text{ ksi} \qquad (Eq.\ C3.1.2.1\text{-}9)$$

$$F_e = \frac{(1.67)(3.90)(0.881)}{2.29}\sqrt{(72.8)(82.4)} = 194 \text{ ksi} \qquad (Eq.\ C3.1.2.1\text{-}4)$$

$$2.78F_y = (2.78)(55) = 153 \text{ ksi}$$

Since $F_e > 2.78\ F_y$, the section is not subject to lateral-torsional buckling.

Calculate the design distortional buckling strength per Section C3.1.4(b).

Since there is no distortional restraint of the bottom flange, take F_d/β from Table II-7

From Table II-7, for the 9CS2.5x059,

$F_d/\beta = 41.2$ ksi

L_{cr} = 25.8 in.

The unbraced length for distortional buckling, L_m, is taken as the distance between the end of the lap and the inflection point.

L_m = 56.3 in. (from above).

$L = \min(L_{cr}, L_m)$

$= \min(25.8,\ 56.3) = 25.8$ in.

The moments at the ends of the segment are:

M_1 = 0.0 kip-ft at the inflection point

M_2 = 6.54 kip-ft at the lap

$\beta = 1.0 \le 1 + 0.4(L/L_m)^{0.7}(1 - M_1/M_2)^{0.7} \le 1.3$ (*Eq.* C3.1.4-11)

$= 1.0 \le 1 + 0.4(25.8/56.3)^{0.7}(1 - 0.0/6.54)^{0.7} \le 1.3$

$= 1.0 \le 1.23 \le 1.3$ therefore, use β = 1.3

$F_d = \beta(F_d/\beta)$

$= 1.23(41.2)$ = 50.7 ksi

Calculate the design distortional buckling strength per Section C3.1.4.

$M_y = S_{fy}F_y$ (*Eq.* C3.1.4-4)

$= (2.29)(55) = 126$ kip-in.

$M_{crd} = S_f F_d$ (*Eq.* C3.1.4-5)

$= 2.29(50.7) = 116$ kip-in.

$\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* C3.1.4-3)

$= \sqrt{126/116} = 1.04 > 0.673$ therefore,

$$M_n = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ C3.1.4\text{-}2)$$

$= (1 - 0.22(1/1.04))(1/1.04)(126)$ = 95.5 kip-in. or 7.96 kip-ft

$\phi_b M_n = (0.90)(7.96)$ = 7.16 kip-ft > 6.54 kip-ft OK (*Eq.* A5.1.1-1)

Calculate the design strength based on the initiation of yielding using Section C3.1.1(a).

$M_n = S_e F_y = (1.89)(55)$ = 104 kip-in. or 8.66 kip-ft (*Eq.* C3.1.1.1-1)

$\phi_b M_n = (0.95)(8.66)$ = 8.23 kip-ft > 6.54 kip-ft OK (*Eq.* A5.1.1-1)

At the location of maximum positive moment

The section is assumed to be fully braced against lateral-torsional buckling.

Calculate the design distortional buckling strength per Section C3.1.4(b).

Calculate the elastic distortional buckling stress, F_d.

$$F_d = \beta \frac{k_{\phi fe} + k_{\phi we} + k_{\phi}}{\tilde{k}_{\phi fg} + \tilde{k}_{\phi wg}} \qquad (Eq.\ C3.1.4\text{-}10)$$

The cross section has a single web and a single edge-stiffened flange as required by Section C3.1.4(b). Consider the contribution of the attached roof panel, which has a rotational stiffnesses, k_ϕ = 0.300 kip-in./rad./in. From Table II-7, for the 9CS2.5x059,

$$k_{\phi fe} = 0.221 \text{ kips}$$

$$\tilde{k}_{\phi fg} = 0.00863 \text{ in.}^2$$

$$k_{\phi we} = 0.209 \text{ kips}$$

$$\tilde{k}_{\phi wg} = 0.00181 \text{ in.}^2$$

Since there is no significant moment gradient in the vicinity of the maximum positive moment, use β = 1.0.

$$F_d = (1.0)\frac{0.221 + 0.209 + 0.300}{0.00863 + 0.00181} = 69.9 \text{ ksi} \qquad (Eq.\ C3.1.4\text{-}10)$$

Calculate the design distortional buckling strength per Section C3.1.4

$$M_y = S_{fy}F_y \qquad (Eq.\ C3.1.4\text{-}4)$$

$$= (2.29)(55) = 126 \text{ kip-in.}$$

$$M_{crd} = S_f F_d \qquad (Eq.\ C3.1.4\text{-}5)$$

$$= (2.29)(69.9) = 160 \text{ kip-in.}$$

$$\lambda_d = \sqrt{M_y / M_{crd}} \qquad (Eq.\ C3.1.4\text{-}3)$$

$$= \sqrt{126/160} = 0.887 > 0.673 \text{ therefore,}$$

$$M_n = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ C3.1.4\text{-}2)$$

$$= (1 - 0.22(1/0.887))(1/0.887)(126) = 107 \text{ kip-in. or } 8.90 \text{ kip-ft}$$

$$\phi_b M_n = (0.90)(8.90) = 8.01 \text{ kip-ft} > 3.16 \text{ kip-ft OK} \qquad (Eq.\ A5.1.1\text{-}1)$$

Calculate the design strength based on the initiation of yielding using Section C3.1.1(a).

$$\phi_b M_n = (0.95)(8.66) = 8.23 \text{ kip-ft} > 3.16 \text{ kip-ft OK} \qquad (Eq.\ A5.1.1\text{-}1)$$

In the region of negative moment between the end of the right lap and the inflection point:

Determine the nominal strength using the distance from the inflection point to the lap as the unbraced length using Section C3.1.2.1(a).

L = $5.04 - 1.25 = 3.79$ ft or 45.5 in.

By inspection, this condition is less severe than the left lap, since the unbraced length is shorter and the required strength is less, therefore the section is OK.

At the negative moment at the center support, the section is assumed to be fully braced: Calculate the nominal strength based on initiation of yielding per Section C3.1.1(a), summing the strength of the two overlapped purlins:

Combined strength of purlins

$\phi_b M_n = (0.95)(8.66 + 8.66) = 16.5$ kip-ft > 7.16 kip-ft OK (*Eq.* A5.1.1-1)

b. Strength for Shear Only (Section C3.2)

Required Strength

By inspection, the load combination 1.2D + 1.6L_r controls:

$V_u = 1.2V_D + 1.6V_{Lr}$

End Span, from left to right:

At left support: $V_u = (1.2)(0.14) + (1.6)(0.86) = 1.54$ kips

At end of right lap: $V_u = (1.2)(0.20) + (1.6)(1.21) = 2.18$ kips

At left side of first interior support: $V_u = (1.2)(0.23) + (1.6)(1.39) = 2.50$ kips

Interior Span, from left to right:

At right side of first interior support: $V_u = (1.2)(0.21) + (1.6)(1.23) = 2.22$ kips

At end of left lap: $V_u = (1.2)(0.16) + (1.6)(0.99) = 1.78$ kips

At end of right lap: $V_u = (1.2)(0.15) + (1.6)(0.90) = 1.62$ kips

At center support: $V_u = (1.2)(0.17) + (1.6)(1.02) = 1.84$ kips

Design Strength

End Span:

At the left support and right lap, t=0.070 in. By inspection the right lap controls.

For t = 0.070 in. and h = 8.485 in., h/t = 121

$1.51\sqrt{Ek_v/F_y} = 1.51\sqrt{(29500)(5.34)/55} = 80.8$

For $\frac{h}{t} > 1.51\sqrt{Ek_v/F_y}$:

$$F_v = \frac{\pi^2 E k_v}{12(1-\mu^2)(h/t)^2} \quad (Eq.\ C3.2.1\text{-}4a)$$

$$= \frac{\pi^2 (29500)(5.34)}{12(1-0.3^2)(8.485/0.070)^2} = 9.69 \text{ ksi}$$

$$V_n = A_w F_v \quad (Eq.\ C3.2.1\text{-}1)$$

$$= (8.485)(0.070)(9.69) = 5.76 \text{ kips}$$

Alternately, V_n can be taken from Table II-1, Beam Properties, C-Sections With Lips, F_y = 55 ksi. For a 9CS2.5x070, V_n is 5.76 kips.

$\phi_v V_n = (0.95)(5.76) = 5.47$ kips > 2.18 kips OK (*Eq.* A5.1.1-1)

At the first interior support, sum the strength of the two overlapped purlins:

For t = 0.059 in. and h = 8.507 in., h/t = 144 > 80.8

$$F_v = \frac{\pi^2 (29500)(5.34)}{12(1-0.3^2)(8.507/0.059)^2} = 6.85 \text{ ksi} \quad (Eq.\ C3.2.1\text{-}4a)$$

$V_n = A_wF_v$ (*Eq.* C3.2.1-1)

$= (8.507)(0.059)(6.85) = 3.44$ kips

Alternately, V_n can be taken from Table II-1, Beam Properties, C-Sections With Lips, F_y = 55 ksi. For a 9CS2.5x059, V_n is 3.44 kips.

For the combined section:

$\phi_vV_n = (0.95)(5.76+3.44) = 8.74$ kips > 2.50 kips OK (*Eq.* A5.1.1-1)

Interior span:

At the first interior support, use the strength computed above:

ϕ_vV_n = 8.74 kip > 2.22 kips OK (*Eq.* A5.1.1-1)

By inspection of the left and right laps, the left lap controls

$\phi_vV_n = (0.95)(3.44) = 3.27$ kips > 1.78 kips OK (*Eq.* A5.1.1-1)

At the center support, sum the strength of the two overlapped purlins:

For the combined section:

$\phi_vV_n = (0.95)(3.44+3.44) = 6.54$ kips > 1.84 kips OK (*Eq.* A5.1.1-1)

c. **Strength for Combined Bending and Shear (Section C3.3.2)**

End Span:

$$\sqrt{\left(\frac{\overline{M}}{\phi_b M_{nxo}}\right)^2 + \left(\frac{\overline{V}}{\phi_v V_n}\right)^2} \le 1.0 \quad (Eq.\ C3.3.2\text{-}1)$$

where

$M_{nxo} = M_n$ calculated based on the initiation of yielding per Section C3.1.1

$\overline{M} = M_u$

$\overline{V} = V_u$

$\phi_b = 0.95$

$\phi_v = 0.95$

At start of lap, t = 0.070 in.

$$\sqrt{\left(\frac{7.34}{(0.95)(11.3)}\right)^2 + \left(\frac{2.18}{(0.95)(5.76)}\right)^2} = 0.791 < 1.0 \text{ OK} \quad (Eq.\ C3.3.2\text{-}1)$$

At first interior support,

$$\sqrt{\left(\frac{12.0}{(0.95)(11.3+8.66)}\right)^2 + \left(\frac{2.50}{(0.95)(5.76+3.44)}\right)^2} = 0.694 < 1.0 \text{ OK} \quad (Eq.\ C3.3.2\text{-}1)$$

Interior Span:

At end of laps, t = 0.059 in. Left lap controls by inspection.

$$\sqrt{\left(\frac{6.54}{(0.95)(8.66)}\right)^2 + \left(\frac{1.78}{(0.95)(3.44)}\right)^2} = 0.964 < 1.0 \text{ OK} \quad (Eq.\ C3.3.2\text{-}1)$$

At center support,

$$\sqrt{\left(\frac{7.16}{(0.95)(8.66+8.66)}\right)^2+\left(\frac{1.84}{(0.95)(3.44+3.44)}\right)^2} = 0.518 < 1.0 \text{ OK} \qquad (Eq.\ C3.3.2\text{-}1)$$

d. Web Crippling Strength (Section C3.4)

Required Strength

By inspection, the load combination D + L_r controls:

$P_u = 1.2P_D + 1.6P_{Lr}$

Supports, from left to right:

At left support: $P_u = (1.2)(0.14) + (1.6)(0.86) = 1.54 \text{ kips}$

At first interior support: $P_u = (1.2)(0.44) + (1.6)(2.63) = 4.74 \text{ kips}$

At center support: $P_u = (1.2)(0.34) + (1.6)(2.03) = 3.66 \text{ kips}$

Design Strength

$$P_n = Ct^2F_y\sin\theta\left(1-C_R\sqrt{\frac{R}{t}}\right)\left(1+C_N\sqrt{\frac{N}{t}}\right)\left(1-C_h\sqrt{\frac{h}{t}}\right) \qquad (Eq.\ C3.4.1\text{-}1)$$

where

$F_y = 55$ ksi

$\theta = 90$ degrees

$R = 0.1875$ in.

N= bearing length = 5.0 in.

At end supports:

h = 8.485 in.

t = 0.070 in.

From Table C3.4.1-2, using the coefficients for the case of:

Fastened to Support/One-Flange Loading or Reaction/End

$C = 4$

$C_R = 0.14$

$C_N = 0.35$

$C_h = 0.02$

$\phi_w = 0.85$

Check Limit: R/t = 0.1875/0.070 = 2.7 < 9 OK

$$P_n = (4)(0.070)^2(55)\sin(90)\left(1-0.14\sqrt{\frac{0.1875}{0.070}}\right)\left(1+0.35\sqrt{\frac{5.0}{0.070}}\right)\left(1-0.02\sqrt{\frac{8.485}{0.070}}\right)$$

$$= 2.57 \text{ kips} \qquad (Eq.\ C3.4.1\text{-}1)$$

$$\phi_w P_n = (0.85)(2.57) = 2.19 \text{ kips} > 1.54 \text{ kips OK} \qquad (Eq.\ A5.1.1\text{-}1)$$

At interior supports:

From Table C3.4.1-2, using the coefficients for the case of:

Fastened to Support/One-Flange Loading or Reaction/Interior

$C = 13$

$C_R = 0.23$

$C_N = 0.14$

C_h = 0.01
ϕ_w = 0.90

for t = 0.070 in.:

Check Limit: $R/t = 0.1875/0.070 = 2.7 < 5$ OK

$$P_n = (13)(0.070)^2(55)\sin(90)\left(1-0.23\sqrt{\frac{0.1875}{0.070}}\right)\left(1+0.14\sqrt{\frac{5.0}{0.070}}\right)\left(1-0.01\sqrt{\frac{8.485}{0.070}}\right)$$

$= 4.25$ kips (*Eq.* C3.4.1-1)

for t = 0.059 in.:

Check Limit: $R/t = 0.1875/0.059 = 3.2 < 5$ OK

$$P_n = (13)(0.059)^2(55)\sin(90)\left(1-0.23\sqrt{\frac{0.1875}{0.059}}\right)\left(1+0.14\sqrt{\frac{5.0}{0.059}}\right)\left(1-0.01\sqrt{\frac{8.507}{0.059}}\right)$$

$= 2.96$ kips (*Eq.* C3.4.1-1)

At first interior support,

$\phi_w P_n = (0.90)(4.25+2.96) = 6.49$ kips > 4.74 kips OK (*Eq.* A5.1.1-1)

At center support,

$\phi_w P_n = (0.90)(2.96+2.96) = 5.33$ kips > 3.66 kips OK (*Eq.* A5.1.1-1)

e. Combined Bending and Web Crippling (Section C3.5.2(a))

$$0.91\left(\frac{\overline{P}}{P_n}\right)+\left(\frac{\overline{M}}{M_{nxo}}\right) \le 1.33\phi \quad (Eq.\ C3.5.2\text{-}1)$$

where

$\overline{P}$ = P_u

$\overline{M}$ = M_u

P_n = the sum of P_n of each purlin at the support

M_{nxo} = the sum of M_n of each purlin at the support calculated based on the initiation of yielding per Section C3.1.1

ϕ = 0.90

$1.33\phi = (1.33)(0.90) = 1.20$

At the first interior support,

$$0.91\left(\frac{4.74}{4.25+2.96}\right)+\left(\frac{12.0}{11.3+8.66}\right) = 1.20 \le 1.20 \quad \text{OK} \quad (Eq.\ C3.5.2\text{-}1)$$

At the center support,

$$0.91\left(\frac{3.66}{2.96+2.96}\right)+\left(\frac{7.16}{8.66+8.66}\right) = 0.976 < 1.20 \quad \text{OK} \quad (Eq.\ C3.5.2\text{-}1)$$

4. Check Uplift Loads

a. Strength for Bending Only (Section D6.1.1)

Required Strength

By inspection, load combination 0.9D + 1.6W controls.

$M_u = 0.9M_D + 1.6M_W$

End Span:

Moment near center of span: $M_u = (0.9)(0.68) + (1.6)(-4.09) = -5.93$ kip-ft

Interior Span:

Moment near center of span: $M_u = (0.9)(0.29) + (1.6)(-1.76) = -2.56$ kip-ft

Design Strength

$M_n = R\,S_e\,F_y$ (*Eq.* D6.1.1-1)

R = 0.60, assuming all 14 conditions of Section D6.1.1 are satisfied

End Span:

For t = 0.070 in.

$M_n = (0.60)(2.47)(55) = 81.5$ kip-in. or 6.79 kip-ft (*Eq.* D6.1.1-1)

$\phi_b M_n = (0.90)(6.79) = 6.11$ kip-ft > 5.93 kip-ft OK (*Eq.* A5.1.1-1)

Interior Span:

For t = 0.059 in.

$M_n = (0.60)(1.89)(55) = 62.4$ kip-in. or 5.20 kip-ft (*Eq.* D6.1.1-1)

$\phi_b M_n = (0.90)(5.20) = 4.68$ kip-ft > 2.56 kip-ft OK (*Eq.* A5.1.1-1)

b. Other Comments

All other regions of the system have their compression flange braced by the roof panel. Since the magnitude of the shears, moments and reactions are less than those of the gravity case, it can be concluded by inspection that the design satisfies the *Specification* criteria for uplift.

5. Anchorage Forces

A check of anchorage forces is required to complete the design. Refer to the *Design Guide for Cold-Formed Steel Purlin Roof Framing Systems*[2] for further information and examples of this check.

[2] AISI publication *Design Guide for Cold-Formed Steel Purlin Roof Framing Systems*

Example II-2: Four Span Continuous Z-Purlins Attached to Through Fastened Roof[3] - ASD

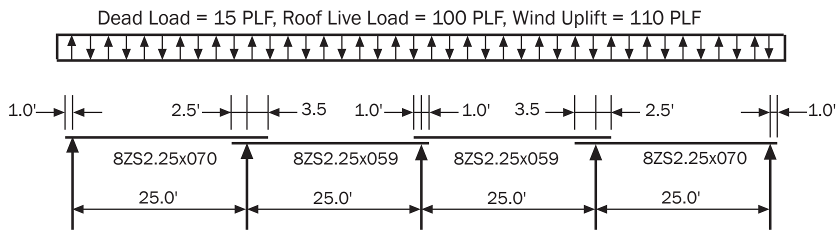


Figure 1 - Spans and Loading

Note: Lap dimensions are to connection points of purlins

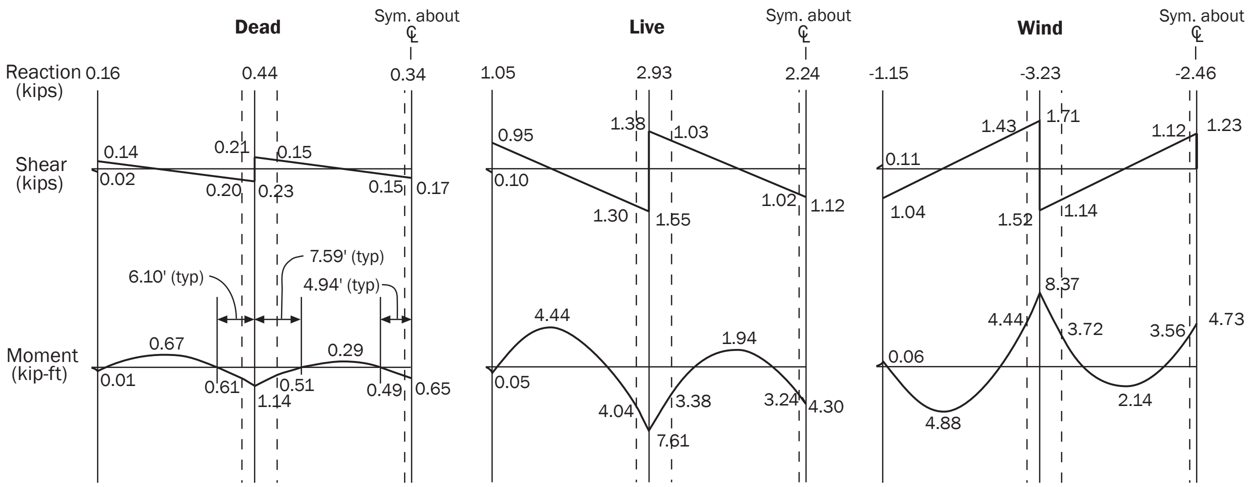


Figure 2 - Reactions, Shears and Moments

Note: Moments and forces are from unfactored nominal loads

Given:

1. Four span Z-purlin system using laps at interior support points to create continuity (see Figures 1 and 2).
2. Roof covering is attached with through fasteners along entire length of purlins.
3. F_y = 55 ksi
4. No discrete bracing lines: anti-roll clips at each support at every fourth purlin line
5. Bottom flanges are bolted to a 0.25 in. thick supporting member with a bearing length of 5 in.
6. The roof covering provides a rotational stiffness to the top flange of the purlins of 0.300 kip-in./rad./in.

Required:

1. Check the design using ASD with ASCE/SEI 7-05 load combinations for:

   (a) Gravity Loads

   (b) Wind Uplift Loads

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[3] For design of purlins supporting standing seam roof, see AISI publication *Design Guide for Cold-Formed Steel Purlin Roof Framing Systems*.

2. Compute the anchorage forces at the supports under gravity loads.

Solution:

1. Assumptions for Analysis and Application of *Specification* Provisions

The *Specification* does not define the methods of analysis to be used; these judgments are the responsibility of the designer. The following assumptions are considered good practice but are not intended to prohibit other approaches:

a. The purlins are connected within the lapped portions in a manner that achieves full continuity between the individual purlin members.

b. It is assumed in the continuous beam analysis that the shear and moment diagrams are based on continuous non-prismatic members between supports in which I_x within the lapped portions is the sum of the I_x of the individual members. Gross values of I_x are used for the beam analysis.

c. The strength within the lapped portions is assumed to be the sum of the strengths of the individual members.

d. It is assumed that the attachment of the roof covering to the purlin provides continuous lateral-torsional support to the top flange.

e. The roof covering provides continuous distortional bracing of 0.300 kip-in./rad./in.

f. For gravity loads, the region at and near the interior supports is assumed to be not subject to lateral-torsional or distortional buckling between the support and the ends of the laps.

g. Under uniform gravity loading, the negative moment region between the end of the lap and the inflection point is assumed to have an unbraced length for lateral-torsional and distortional buckling equal to the distance from the end of the lap to the inflection point.

h. Since the loading, geometry and materials are symmetrical, only the first two spans are checked.

2. Calculation of Section Properties

Based on the design procedures illustrated in Examples I-3 and I-10 and Table I-4 and II-4 of the *Design Manual*, the following section properties have been obtained for the two Z-sections:

Section	8ZS2.25x059	8ZS2.25x070
D (in.)	8.0	8.0
t (in.)	0.059	0.070
R (in.)	0.1875	0.1875
I_x (in.^{4})	7.76	9.18
S_f (in.^{3})	1.94	2.30
S_e (in.^{3})	1.80	2.25
I_y (in.^{4})	1.08	1.28

3. Check Gravity Loads

a. Strength for Bending Only (Section C3.1)

Required Allowable Strength

By inspection, the load combination D + L_r controls:

$M = M_D + M_{Lr}$

Overhang:

Maximum negative moment:	M = 0.01 + 0.05 = 0.06 kip-ft

End Span, from left to right:

Maximum positive moment:	M = 0.67 + 4.44 = 5.11 kip-ft
Negative moment at end of right lap:	M = 0.61 + 4.04 = 4.65 kip-ft
Negative moment at support:	M = 1.14 + 7.61 = 8.75 kip-ft

Interior span, from left to right:

Negative moment at end of left lap:	M = 0.51 + 3.38 = 3.89 kip-ft
Maximum positive moment:	M = 0.29 + 1.94 = 2.23 kip-ft
Negative moment at end of right lap:	M = 0.49 + 3.24 = 3.73 kip-ft
Negative moment at center support:	M = 0.65 + 4.30 = 4.95 kip-ft

Allowable Flexural Strength

Compute the lowest of the applicable flexural strengths from Sections C3.1.1 (initiation of yielding), C3.1.2 (lateral-torsional buckling), C3.1.4 (distortional buckling) and D6.1.1 (flexural strength of members having tension flange through-fastened to deck or sheathing).

Overhang:

By inspection, due to the short unbraced length and insignificant bending moment, the overhang is acceptable. Calculations for the negative moment region of the end span below demonstrate that the allowable bending strength of the section greatly exceeds the required strength at the overhang.

End Span:

At location of maximum positive moment

The section is assumed to be fully braced against lateral-torsional buckling, but distortional buckling and yielding strengths must be calculated.

Calculate the allowable distortional buckling strength per Section C3.1.4(b).

A conservative distortional buckling strength can be calculated using Section C3.1.4(a) for members meeting the limitations of that section. In this case, use the more accurate provisions of Section C3.1.4(b) to take advantage of the stiffness provided by the roof covering.

Calculate the elastic distortional buckling stress, F_d.

The cross section has a single web and a single edge-stiffened flange as required by Section C3.1.4(b). Consider the contribution of the attached roof panel, which has a rotational stiffnesses, k_ϕ = 0.300 kip-in./rad./in. From Table II-9,

$$k_{\phi fe} = 0.437 \text{ kips}$$

$$\tilde{k}_{\phi fg} = 0.0153 \text{ in.}^2$$

$$k_{\phi we} = 0.402 \text{ kips}$$

$$\tilde{k}_{\phi wg} = 0.00231 \text{ in.}^2$$

Since there is no significant moment gradient in the vicinity of the maximum positive moment, use β = 1.0.

$$F_d = \beta \frac{k_{\phi fe} + k_{\phi we} + k_{\phi}}{\tilde{k}_{\phi fg} + \tilde{k}_{\phi wg}} \quad (Eq.\ C3.1.4\text{-}10)$$

$$= (1.0)\frac{0.437 + 0.402 + 0.300}{0.0153 + 0.00231} = 64.7 \text{ ksi}$$

Calculate the allowable distortional buckling strength per Section C3.1.4

$$M_y = S_{fy}F_y \quad (Eq.\ C3.1.4\text{-}4)$$

$$= (2.30)(55) = 127 \text{ kip-in.}$$

$$M_{crd} = S_f F_d \quad (Eq.\ C3.1.4\text{-}5)$$

$$= (2.30)(64.7) = 149 \text{ kip-in.}$$

$$\lambda_d = \sqrt{M_y / M_{crd}} \quad (Eq.\ C3.1.4\text{-}3)$$

$$= \sqrt{127/149} = 0.923 > 0.673 \text{ therefore,}$$

$$M_n = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ C3.1.4\text{-}2)$$

$$= \left(1 - 0.22(1/0.923)\right)(1/0.923)(127) = 105 \text{ kip-in. or } 8.73 \text{ kip-ft}$$

$$\frac{M_n}{\Omega_b} = \frac{8.73}{1.67} = 5.23 \text{ kip-ft} > 5.11 \text{ kip-ft OK} \quad (Eq.\ A4.1.1\text{-}1)$$

Calculate the allowable strength based on initiation of yielding per Section C3.1.1(a).

For the exterior purlin, t = 0.070 in.

$$M_n = S_e F_y = (2.25)(55) = 124 \text{ kip-in.} = 10.3 \text{ kip-ft} \quad (Eq.\ C3.1.1\text{-}1)$$

$$\frac{M_n}{\Omega_b} = \frac{10.3}{1.67} = 6.17 \text{ kip-ft} > 5.11 \text{ kip-ft OK} \quad (Eq.\ A4.1.1\text{-}1)$$

In the region of negative moment between the end of the lap and the inflection point:

Calculate the allowable lateral-torsional buckling strength per Section C3.1.2.1(b).

Determine the nominal strength using the distance from the inflection point to the end of the lap as the unbraced length.

$$F_e = \frac{C_b \pi^2 E d I_{yc}}{2S_f\left(K_y L_y\right)^2} \quad (Eq.\ C3.1.2.1\text{-}15)$$

$$L_y = 6.10 - 2.50 = 3.60 \text{ ft} = 43.2 \text{ in.}$$

$$K_y = 1.0$$

$$I_{yc} = \frac{I_y}{2} = \frac{1.28}{2} = 0.640 \text{ in.}^4$$

$C_b = 1.67$ (Conservatively assumes linear moment diagram in this region)

$$F_e = \frac{1.67\pi^2(29500)(8.0)(0.640)}{(2)(2.30)\left(1.0(43.2)\right)^2} = 290 \text{ ksi} \quad (Eq.\ C3.1.2.1\text{-}15)$$

$$2.78F_y = (2.78)(55) = 153 \text{ ksi}$$

Since F_e > $2.78F_y$, the unbraced length is not subject to lateral-torsional buckling.

Calculate the allowable distortional buckling strength per Section C3.1.4(b).

$$F_d = \beta \frac{k_{\phi fe} + k_{\phi we} + k_{\phi}}{\tilde{k}_{\phi fg} + \tilde{k}_{\phi wg}} \qquad (Eq.\ C3.1.4\text{-}10)$$

Since there is no distortional restraint of the bottom flange, $k_\phi = 0$

$$F_d/\beta = \frac{0.437 + 0.402 + 0.0}{0.0153 + 0.00231} = 47.6 \text{ ksi}$$

Alternately, F_d/β, for the case where $k_\phi = 0$, may be taken from Table II-9:

$$F_d/\beta = 47.7 \text{ ksi}$$

From Table II-9,

L_{cr} = 20.8 in.

The unbraced length for distortional buckling is taken as the distance between the end of the lap and the inflection point.

L_m = 43.2 in. (from above).

$$L = \min(L_{cr}, L_m)$$

$$= \min(20.8, 43.2) = 20.8 \text{ in.}$$

The moments at the ends of the segment are:

M_1 = 0.0 kip-ft at the inflection point

M_2 = 4.65 kip-ft at the lap

$$\beta = 1.0 \le 1 + 0.4(L/L_m)^{0.7}(1 - M_1/M_2)^{0.7} \le 1.3 \qquad (Eq.\ C3.1.4\text{-}7)$$

$$= 1.0 \le 1 + 0.4(20.8/43.2)^{0.7}(1 - 0.0/4.65)^{0.7} \le 1.3$$

$$= 1.0 \le 1.24 \le 1.3 \text{ therefore, use } \beta = 1.24$$

$$F_d = \beta(F_d/\beta)$$

$$= 1.24(47.6) = 59.0 \text{ ksi}$$

Calculate the allowable distortional buckling strength per Section C3.1.4

$$M_y = S_{fy}F_y \qquad (Eq.\ C3.1.4\text{-}4)$$

$$= (2.30)(55) = 127 \text{ kip-in.}$$

$$M_{crd} = S_f F_d \qquad (Eq.\ C3.1.4\text{-}5)$$

$$= (2.30)(59.0) = 136 \text{ kip-in.}$$

$$\lambda_d = \sqrt{M_y/M_{crd}} \qquad (Eq.\ C3.1.4\text{-}3)$$

$$= \sqrt{127/136} = 0.966 > 0.673 \text{ therefore,}$$

$$M_n = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ C3.1.4\text{-}2)$$

$$= (1 - 0.22(1/0.966))(1/0.966)(127) = 102 \text{ kip-in. or } 8.46 \text{ kip-ft}$$

$$\frac{M_n}{\Omega_b} = \frac{8.46}{1.67} = 5.07 \text{ kip-ft} > 4.65 \text{ kip-ft OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

Calculate the allowable strength based on the initiation of yielding using Section C3.1.1(a).

$$M_n = S_eF_y \qquad (Eq.\ C3.1.1\text{-}1)$$

$$= (2.25)(55.0) = 124 \text{ kip-in. or } 10.3 \text{ kip-ft}$$

$$\frac{M_n}{\Omega_b} = \frac{10.3}{1.67} = 6.17 \text{ kip-ft} > 4.65 \text{ kip-ft OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

In the lapped region over the support:

The lapped section is assumed to be sufficiently restrained against lateral-torsional buckling and distortional buckling.

Calculate the allowable flexural strength based on initiation of yielding per Section C3.1.1(a), summing the strength of the two overlapped purlins:

For the exterior purlin, t = 0.070 in.

$$M_n = S_eF_y = (2.25)(55) = 124 \text{ kip-in. or } 10.3 \text{ kip-ft} \qquad (Eq.\ C3.1.1\text{-}1)$$

For the interior purlin, t = 0.059 in.

$$M_n = S_eF_y = (1.80)(55) = 99.0 \text{ kip-in. or } 8.25 \text{ kip-ft} \qquad (Eq.\ C3.1.1\text{-}1)$$

Combined strength of purlins

$$\frac{M_n}{\Omega_b} = \frac{10.3 + 8.25}{1.67} = 11.1 \text{ kip-ft} > 8.75 \text{ kip-ft OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

Interior Span:

In the region of negative moment between the end of the left lap and the inflection point:

Calculate the allowable lateral-torsional buckling strength per Section C3.1.2.1(b).

Determine the nominal strength using the distance from the inflection point to the end of the lap as the unbraced length using Section C3.1.2.1(b).

$$L_y = 7.59 - 3.50 = 4.09 \text{ ft or } 49.1 \text{ in.}$$

$$K_y = 1.0$$

$$I_{yc} = \frac{I_y}{2} = \frac{1.08}{2} = 0.540 \text{ in.}^4$$

C_b = 1.67 (Conservatively assumes linear moment diagram in this region)

$$F_e = \frac{1.67\pi^2(29500)(8.0)(0.540)}{(2)1.94\left(1.0(49.1)\right)^2} = 225 \text{ ksi} \qquad (Eq.\ C3.1.2.1\text{-}15)$$

Since $F_e > 2.78F_y$, the segment is not subject to lateral-torsional buckling.

Calculate the allowable distortional buckling strength per Section C3.1.4(b).

Since there is no distortional restraint of the bottom flange, take F_d/β from Table II-9

From Table II-9, for the ZS2.25x059,

$$F_d/\beta = 38.6 \text{ ksi}$$

L_{cr} = 22.4 in.

The unbraced length for distortional buckling is taken as the distance between the end of the lap and the inflection point.

L_m = 49.1 in. (from above).

$$L = \min(L_{cr}, L_m)$$

$$= \min(22.4, 49.1) = 22.4 \text{ in.}$$

The moments at the ends of the segment are:

M_1 = 0.0 kip-ft at the inflection point

M_2 = 3.89 kip-ft at the lap

$$\beta = 1.0 \le 1 + 0.4(L/L_m)^{0.7}(1 - M_1/M_2)^{0.7} \le 1.3 \qquad (Eq.\ C3.1.4\text{-}7)$$

$$= 1.0 \le 1 + 0.4(22.4/49.1)^{0.7}(1 - 0.0/3.89)^{0.7} \le 1.3$$

$$= 1.0 \le 1.23 \le 1.3 \text{ therefore, use } \beta = 1.23$$

$$F_d = \beta(F_d/\beta)$$

$$= 1.23(38.6) = 47.5 \text{ ksi}$$

Calculate the allowable distortional buckling strength per Section C3.1.4

$$M_y = S_{fy}F_y \qquad (Eq.\ C3.1.4\text{-}4)$$

$$= (1.94)(55) = 107 \text{ kip-in.}$$

$$M_{crd} = S_f F_d \qquad (Eq.\ C3.1.4\text{-}5)$$

$$= (1.94)(47.5) = 92.2 \text{ kip-in.}$$

$$\lambda_d = \sqrt{M_y/M_{crd}} \qquad (Eq.\ C3.1.4\text{-}3)$$

$$= \sqrt{107/92.2} = 1.08 > 0.673 \text{ therefore,}$$

$$M_n = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ C3.1.4\text{-}2)$$

$$= (1 - 0.22(1/1.08))(1/1.08)(107) = 78.9 \text{ kip-in. or } 6.57 \text{ kip-ft}$$

$$\frac{M_n}{\Omega_b} = \frac{6.57}{1.67} = 3.93 \text{ kip-ft} > 3.89 \text{ kip-ft OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

Calculate the allowable strength based on the initiation of yielding using Section C3.1.1(a).

$$M_n = S_e F_y = (1.80)(55.0) = 99.0 \text{ kip-in. or } 8.25 \text{ kip-ft} \qquad (Eq.\ C3.1.1\text{-}1)$$

$$\frac{M_n}{\Omega_b} = \frac{8.25}{1.67} = 4.99 \text{ kip-ft} > 3.89 \text{ kip-ft OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

At the location of maximum positive moment:

The section is assumed to be fully braced against lateral-torsional buckling.

Calculate the allowable distortional buckling strength per Section C3.1.4(b).

Calculate the elastic distortional buckling stress, F_d.

The cross section has a single web and a single edge-stiffened flange as required by Section C3.1.4(b). Consider the contribution of the attached roof panel, which has a rotational stiffnesses, k_ϕ = 0.300 kip-in./rad./in. From Table II-9,

$$k_{\phi fe} = 0.254 \text{ kips}$$

$$\tilde{k}_{\phi fg} = 0.0110 \text{ in.}^2$$

$$k_{\phi we} = 0.236 \text{ kips}$$

$$\tilde{k}_{\phi wg} = 0.00168 \text{ in.}^2$$

Since there is no significant moment gradient in the vicinity of the maximum positive moment, use β = 1.0.

$$F_d = \beta \frac{k_{\phi fe} + k_{\phi we} + k_\phi}{\tilde{k}_{\phi fg} + \tilde{k}_{\phi wg}} \qquad (Eq.\ C3.1.4\text{-}10)$$

$$= (1.0)\frac{0.254 + 0.236 + 0.300}{0.0110 + 0.00168} = 62.3 \text{ ksi}$$

Calculate the allowable distortional buckling strength per Section C3.1.4

$$M_y = S_{fy}F_y \qquad (Eq.\ C3.1.4\text{-}4)$$

$$= (1.94)(55) = 107 \text{ kip-in.}$$

$$M_{crd} = S_f F_d \qquad (Eq.\ C3.1.4\text{-}5)$$

$$= (1.94)(62.3) = 121 \text{ kip-in.}$$

$$\lambda_d = \sqrt{M_y / M_{crd}} \qquad (Eq.\ C3.1.4\text{-}3)$$

$$= \sqrt{107/121} = 0.940 > 0.673 \text{ therefore,}$$

$$M_n = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ C3.1.4\text{-}2)$$

$$= \left(1 - 0.22(1/0.940)\right)(1/0.940)(107) = 87.2 \text{ kip-in. or } 7.27 \text{ kip-ft}$$

$$\frac{M_n}{\Omega_b} = \frac{7.27}{1.67} = 4.35 \text{ kip-ft} > 2.23 \text{ kip-ft OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

Calculate the allowable strength based on initiation of yielding per Section C3.1.1(a).

$$\frac{M_n}{\Omega_b} = \frac{8.25}{1.67} = 4.94 \text{ kip-ft} > 2.23 \text{ kip-ft OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

In the region of negative moment between the end of the right lap and the inflection point:

Determine the nominal strength using the distance from the inflection point to the end of the lap as the unbraced length per Section C3.1.2.1(b).

$$L = 4.94 - 1.00 = 3.94 \text{ ft or } 47.3 \text{ in.}$$

By inspection, this condition is less severe than that adjacent to the left lap, since the unbraced length is shorter and the required strength is less; therefore, the section is OK.

In the lapped region over the center support, the section is assumed to be fully braced:

Use nominal strength based on initiation of yielding per Section C3.1.1(a), summing the strength of the two overlapped purlins:

Combined strength of purlins

$$\frac{M_n}{\Omega_b} = \frac{8.25 + 8.25}{1.67} = 9.88 \text{ kip-ft} > 4.95 \text{ kip-ft OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

b. Strength for Shear Only (Section C3.2)

Required Allowable Strength

By inspection, the load combination D + L_r controls:

$V = V_D + V_{Lr}$

Overhang By inspection, the overhang does not control.

End Span, from left to right:

At left support:	V = 0.14 + 0.95 = 1.09 kips
At end of right lap:	V = 0.20 + 1.30 = 1.50 kips
At left side of first interior support:	V = 0.23 + 1.55 = 1.78 kips

Interior Span, from left to right:

At right side of first interior support:	V = 0.21 + 1.38 = 1.59 kips
At end of left lap:	V = 0.15 + 1.03 = 1.18 kips
At end of right lap:	V = 0.15 + 1.02 = 1.17 kips
At center support:	V = 0.17 + 1.12 = 1.29 kips

Allowable Strength

End Span:

At the left support and right lap, t = 0.070 in. By inspection the right lap controls.

For t = 0.070 in. and h = 7.485 in., h/t = 107

$$\frac{h}{t} > 1.51\sqrt{Ek_v/F_y} = 1.51\sqrt{(29500)(5.34)/55} = 80.8$$

$$F_v = \frac{\pi^2 E k_v}{12(1-\mu^2)(h/t)^2} \qquad (Eq.\ C3.2.1\text{-}4a)$$

$$= \frac{\pi^2(29500)(5.34)}{12(1-0.3^2)(7.485/0.070)^2} = 12.5 \text{ ksi}$$

$$V_n = A_w F_v \qquad (Eq.\ C3.2.1\text{-}1)$$

$$= (7.485)(0.070)(12.5) = 6.55 \text{ kips}$$

Alternately, V_n can be taken from Table II-4, Beam Properties, Z-Sections With Lips, F_y = 55 ksi. For a 8ZS2.25x070, V_n is 6.52 kips.

$$\frac{V_n}{\Omega_v} = \frac{6.55}{1.60} = 4.09 \text{ kips} > 1.50 \text{ kips OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

At the left side of first interior support, sum the strength of the two overlapped purlins:

For t = 0.059 in. and h = 7.507 in., h/t = 127

$$\frac{h}{t} > 1.51\sqrt{Ek_v/F_y} = 80.8$$

$$F_v = \frac{\pi^2(29500)(5.34)}{12(1-0.3^2)(7.507/0.059)^2} = 8.80 \text{ ksi} \quad (Eq.\ C3.2.1\text{-}4a)$$

$$V_n = A_w F_v \quad (Eq.\ C3.2.1\text{-}1)$$

$$= (7.507)(0.059)(8.80) = 3.90 \text{ kips}$$

Alternately, V_n can be taken from Table II-4, Beam Properties, Z-Sections With Lips, F_y = 55 ksi. For a 8ZS2.25x059, V_n is 3.90 kips.

For the combined section:

$$\frac{V_n}{\Omega_v} = \frac{3.90+6.55}{1.60} = 6.53 \text{ kips} > 1.78 \text{ kips OK} \quad (Eq.\ A4.1.1\text{-}1)$$

Interior span:

At the right side of the first interior support, use the strength computed above.

$$\frac{V_n}{\Omega_v} = 6.53\text{kips} > 1.59 \text{ kips OK} \quad (Eq.\ A4.1.1\text{-}1)$$

By inspection of the left and right laps, the left lap controls.

$$\frac{V_n}{\Omega_v} = \frac{3.90}{1.60} = 2.44 \text{ kips} > 1.18 \text{ kips OK} \quad (Eq.\ A4.1.1\text{-}1)$$

At the center support, sum the strength of the two overlapped purlins:

For the combined section:

$$\frac{V_n}{\Omega_v} = \frac{3.90+3.90}{1.60} = 4.88 \text{ kips} > 1.29 \text{ kips OK} \quad (Eq.\ A4.1.1\text{-}1)$$

c. Strength for Combined Bending and Shear (Section C3.3.1)

End Span:

$$\sqrt{\left(\frac{\Omega_b M}{M_{nxo}}\right)^2 + \left(\frac{\Omega_v V}{V_n}\right)^2} \le 1.0 \quad (Eq.\ C3.3.1\text{-}1)$$

where

M_{nxo}= M_n, calculated based on the initiation of yielding per Section C3.1.1

Ω_b = 1.67

Ω_v = 1.60

To the left of the right lap, t = 0.070 in.

$$\sqrt{\left(\frac{(1.67)(4.65)}{10.3}\right)^2 + \left(\frac{(1.60)(1.50)}{6.55}\right)^2} = 0.838 < 1.0 \text{ OK} \quad (Eq.\ C3.3.1\text{-}1)$$

At first interior support,

$$\sqrt{\left(\frac{(1.67)(8.75)}{10.3+8.25}\right)^2 + \left(\frac{(1.60)(1.78)}{6.55+3.90}\right)^2} = 0.834 < 1.0 \text{ OK} \quad (Eq.\ C3.3.1\text{-}1)$$

Interior Span:

At ends of laps, t = 0.059 in. Left lap controls by inspection.

$$\sqrt{\left(\frac{(1.67)(3.89)}{8.25}\right)^2+\left(\frac{(1.60)(1.18)}{3.90}\right)^2}=0.924<1.0 \text{ OK} \qquad (Eq.\ C3.3.1\text{-}1)$$

At center support,

$$\sqrt{\left(\frac{(1.67)(4.95)}{8.25+8.25}\right)^2+\left(\frac{(1.60)(1.29)}{3.90+3.90}\right)^2}=0.567<1.0 \text{ OK} \qquad (Eq.\ C3.3.1\text{-}1)$$

d. Web Crippling Strength (Section C3.4)

Required Allowable Strength

By inspection, the load combination D + L_r controls:

$P = P_D + P_{Lr}$

Supports, from left to right:

At left support: $P = 0.16 + 1.05 = 1.21$ kips
At first interior support: $P = 0.44 + 2.93 = 3.37$ kips
At center support: $P = 0.34 + 2.24 = 2.58$ kips

Allowable Strength

$$P_n = Ct^2F_y\sin\theta\left(1-C_R\sqrt{\frac{R}{t}}\right)\left(1+C_N\sqrt{\frac{N}{t}}\right)\left(1-C_h\sqrt{\frac{h}{t}}\right) \qquad (Eq.\ C3.4.1\text{-}1)$$

where

F_y = 55 ksi
θ = 90 degrees
R = 0.1875 in.
N = bearing length = 5.0 in.

At outside supports:

h = 7.485 in.
t = 0.070 in.

From Table C3.4.1-3, using the coefficients for the case of:

Fastened to Support/One-Flange Loading or Reaction/End

C = 4
C_R = 0.14
C_N = 0.35
C_h = 0.02
Ω_w = 1.75

Check Limit: R/t = 0.1875/0.070 = 2.7 < 9 OK

$$P_n = (4)(0.070)^2(55)\sin(90)\left(1-0.14\sqrt{\frac{0.1875}{0.070}}\right)\left(1+0.35\sqrt{\frac{5.0}{0.070}}\right)\left(1-0.02\sqrt{\frac{7.485}{0.070}}\right)$$

$$= 2.61 \text{ kips} \qquad (Eq.\ C3.4.1\text{-}1)$$

Alternately, P_n can be taken from Table II-15, Web Crippling, Z-Sections With Lips. Using values for Fastened, Case A, for a 8ZS2.25x070 with a yield stress of 55 ksi, P_n can be interpolated as:

$P_n \quad = 0.5(2.40 + 2.79) = 2.60$ kips.

Calculate strength increase from overhang

$$P_{nc} = \alpha P_n \qquad (Eq.\ 3.4.1\text{-}2)$$

where

$$\alpha = \frac{1.34(L_o/h)^{0.26}}{0.009(h/t)+0.03} \ge 1.0 \qquad (Eq.\ 3.4.1\text{-}3)$$

$$= \frac{1.34(12.0/7.485)^{0.26}}{0.009(7.485/0.070)+0.03} = 1.53$$

Using the value of P_n calculated from *Eq*. C3.4.1-1 above,

$$P_{nc} = (1.53)(2.61) = 3.99 \text{ kips} \qquad (Eq.\ 3.4.1\text{-}2)$$

Check upper limit of interior one-flange loading. From Table C3.4.1-3, using the coefficients for the case of:

Fastened to Support/One-Flange Loading or Reaction/Interior

$C \quad = 13$
$C_R = 0.23$
$C_N = 0.14$
$C_h = 0.01$
$\Omega_w = 1.65$

for t = 0.070 in.:

Check Limit: $R/t = 0.1875/0.070 = 2.7 < 5.5$ OK

$$P_n = (13)(0.070)^2(55)\sin(90)\left(1-0.23\sqrt{\frac{0.1875}{0.070}}\right)\left(1+0.14\sqrt{\frac{5.0}{0.070}}\right)\left(1-0.01\sqrt{\frac{7.485}{0.070}}\right)$$

$$= 4.28 \text{ kips} \qquad (Eq.\ C3.4.1\text{-}1)$$

Alternately, P_n can be taken from Table II-15, Web Crippling, Z-Sections With Lips. Using Case B, for a 8ZS2.25x070 with a yield stress of 55 ksi P_n can be interpolated as:

$P_n = 0.5(4.03 + 4.50) = 4.27 \text{ kips} > 3.99 \text{ kips}$ OK

$$\frac{P_{nc}}{\Omega_w} = \frac{3.99}{1.75} = 2.28 \text{ kips} > 1.21 \text{ kips OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

At interior supports:

From Table C3.4.1-3, using the coefficients for the case of:

Fastened to Support/One-Flange Loading or Reaction/Interior

$C \quad = 13$
$C_R = 0.23$
$C_N = 0.14$
$C_h = 0.01$
$\Omega_w = 1.65$

for t = 0.070 in.,

P_n = 4.28 kips, calculated above

for t = 0.059 in.,

Check Limit: $R/t = 0.1875/0.059 = 3.2 < 5.5$ OK

$$P_n = (13)(0.059)^2(55)\sin(90)\left(1-0.23\sqrt{\frac{0.1875}{0.059}}\right)\left(1+0.14\sqrt{\frac{5.0}{0.059}}\right)\left(1-0.01\sqrt{\frac{7.507}{0.059}}\right)$$

= 2.98 kips (*Eq.* C3.4.1-1)

Alternately, P_n can be taken from Table II-15, Web Crippling, Z-Sections With Lips. Using values for Fastened, Case B, for a 8ZS2.25x059 with a yield stress of 55 ksi P_n can be interpolated as:

$P_n = 0.5(2.80+3.14) = 2.97$ kips.

Using the values of P_n calculated from *Eq.* C3.4.1-1 above,

At first interior support,

$$\frac{P_n}{\Omega_w} = \frac{4.28+2.98}{1.65} = 4.40 \text{ kips} > 3.37 \text{ kips OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

At center support,

$$\frac{P_n}{\Omega_w} = \frac{2.98+2.98}{1.65} = 3.61 \text{ kips} > 2.58 \text{ kips OK} \qquad (Eq.\ A4.1.1\text{-}1)$$

e. Combined Bending and Web Crippling (Section C3.5.1(c))

$$0.86\left(\frac{P}{P_n}\right)+\left(\frac{M}{M_{nxo}}\right) \le \frac{1.65}{\Omega} \qquad (Eq.\ C3.5.1\text{-}3)$$

where

M_{nxo} = the sum of M_n of each purlin at the support calculated based on the initiation of yielding per Section C3.1.1

P_n = the sum of P_n of each purlin at the support

Ω = 1.70

Check the limits for the controlling (thinner) section:

h/t = 7.507/0.059 = 127 < 150 OK

N/t = 5.0/0.059 = 85 < 140 OK

F_y = 55 ksi < 70 ksi OK

R/t = 0.1875/0.059 = 3.2 < 5.5 OK

t_{max}/t_{min} = 0.070/0.059 = 1.19 < 1.3 OK

All other limits are assumed to be satisfied as well.

At the first interior support,

$$0.86\left(\frac{3.37}{4.28+2.98}\right)+\left(\frac{8.75}{10.3+8.25}\right) = 0.871 < \frac{1.65}{1.70} = 0.971 \text{ OK} \qquad (Eq.\ C3.5.1\text{-}3)$$

At the center support,

$$0.86\left(\frac{2.58}{2.98+2.98}\right)+\left(\frac{4.95}{8.25+8.25}\right) = 0.672 < 0.971 \text{ OK} \qquad (Eq.\ C3.5.1\text{-}3)$$

4. Check Uplift Loads

a. Strength for Bending Only (Section D6.1.1)

Required Allowable Strength

By inspection, the load combination D + W controls.

$M = M_D + M_w$

End Span:

Maximum Negative Moment: $M = 0.67 - 4.88 = -4.21$ kip-ft

Interior Span:

Maximum Negative Moment: $M = 0.29 - 2.14 = -1.85$ kip-ft

Allowable Strength

$$M_n = RS_eF_y \quad (Eq.\ D6.1.1\text{-}1)$$

$R = 0.70$, assuming all 14 conditions of Section D6.1.1 are satisfied

End Span:

For t = 0.070 in.

$$M_n = (0.70)(2.26)(55) = 87.0 \text{ kip-in. or } 7.25 \text{ kip-ft} \quad (Eq.\ D6.1.1\text{-}1)$$

$$\frac{M_n}{\Omega_b} = \frac{7.25}{1.67} = 4.34 \text{ kip-ft} > 4.21 \text{ kip-ft OK} \quad (Eq.\ A4.1.1\text{-}1)$$

Interior Span:

For t = 0.059 in.

$$M_n = (0.70)(1.82)(55) = 70.1 \text{ kip-in. or } 5.84 \text{ kip-ft} \quad (Eq.\ D6.1.1\text{-}1)$$

$$\frac{M_n}{\Omega_b} = \frac{5.84}{1.67} = 3.50 \text{ kip-ft} > 1.85 \text{ kip-ft OK} \quad (Eq.\ A4.1.1\text{-}1)$$

b. Other Comments

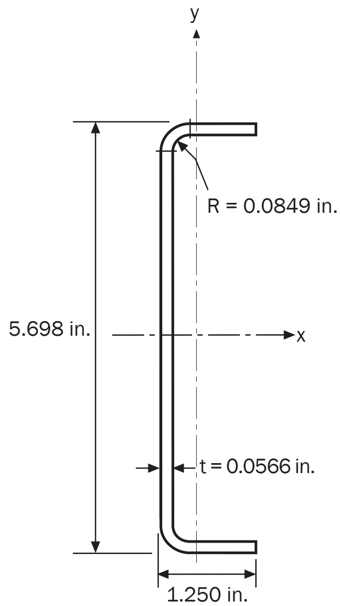
All other regions of the system have their compression flange braced by the roof panel. Since the magnitude of the shears, moments and reactions are less than those of the gravity case, it can be concluded that the design satisfies the *Specification* criteria for uplift.

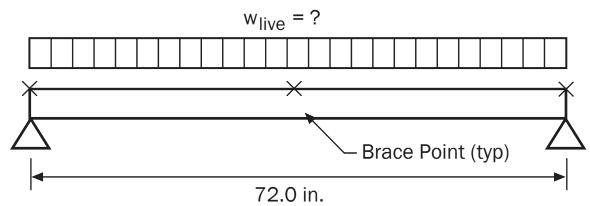
5. Anchorage Forces under Gravity Loads

A check of anchorage forces is required to complete the design. Refer to the *Design Guide for Cold Formed Steel Purlin Roof Framing Systems*[4] for further information and examples of this check.

[4] AISI publication, *Design Guide for Cold Formed Steel Purlin Roof Framing Systems*

Example II-3: C-Section Without Lips Braced At Mid-span





Given:

1. Steel: F_y = 33 ksi
2. Section: SSMA Track 550T125-54 as shown in sketch above.
3. Simple span of 72.0 in.
4. Member is braced against twisting and lateral deflection at mid-span and ends. Ends are reinforced against crippling
5. Member is loaded through the shear center and parallel to the web.
6. Gross section properties from Table I-3:

   t = 0.0566 in.
   R = 0.0849 in.
   A = 0.452 in.^{2}
   S_x = 0.668 in.^{3}
   r_y = 0.342 in.
   r_o = 2.15 in.
   J = 0.000483 in.^{4}
   C_w = 0.315 in.^{6}

Required:

1. Calculate the largest permitted uniformly distributed service load, w_{live}, assuming a negligible dead load, using ASD and LRFD. Compute w_{live} based on flexural strength and check shear.

Solution:

The beam is subject to lateral-torsional buckling, but not subject to distortional buckling, because all of the buckling modes involving change in the cross-sectional shape are local buckling modes (flange local buckling and web local buckling).

1. Lateral-torsional buckling strength (Section C3.1.2.1)

For singly symmetric sections bent about the axis of symmetry,

$$F_e = \frac{C_b r_o A}{S_f}\sqrt{\sigma_{ey}\sigma_t} \qquad (Eq.\ C3.1.2.1\text{-}4)$$

$$\sigma_{ey} = \frac{\pi^2 E}{\left(K_y L_y / r_y\right)^2} \qquad (Eq.\ C3.1.2.1\text{-}8)$$

$$= \frac{\pi^2(29500)}{\left((1.0)(36.0)/0.342\right)^2}$$

$$= 26.28 \text{ ksi}$$

$$\sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2}\right] \qquad (Eq.\ C3.1.2.1\text{-}9)$$

$$\sigma_t = \frac{1}{(0.452)(2.15)^2}\left[(11300)(0.000483) + \frac{\pi^2(29500)(0.315)}{\left[(1.0)(36.0)\right]^2}\right]$$

$$= 36.48 \text{ ksi}$$

Calculate C_b assuming a unit uniform loading

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \qquad (Eq.\ C3.1.2.1\text{-}6)$$

$$M_{max} = \frac{wL^2}{8} = \frac{(1.0)(72.0)^2}{8} = 648.0 \text{ kip-in. at brace point}$$

$$M_A = \frac{7wL^2}{128} = \frac{(7)(1.0)(72.0)^2}{128} = 283.5 \text{ kip-in. at 1/4 point of unbraced segment}$$

$$M_B = \frac{3wL^2}{32} = \frac{(3)(1.0)(72.0)^2}{32} = 486.0 \text{ kip-in. at center point of unbraced segment}$$

$$M_C = \frac{15wL^2}{128} = \frac{(15)(1.0)(72.0)^2}{128} = 607.5 \text{ kip-in. at 3/4 point of unbraced segment}$$

$$C_b = \frac{(12.5)(648.0)}{(2.5)(648.0)+(3)(283.5)+(4)(486.0)+(3)(607.5)} \qquad (Eq.\ C3.1.2.1\text{-}6)$$

$$= 1.30$$

$$F_e = \frac{(1.30)(2.15)(0.452)}{0.668}\sqrt{(26.28)(36.48)} = 58.6 \text{ ksi} \qquad (Eq.\ C3.1.2.1\text{-}4)$$

$0.56F_y = (0.56)(33.0) = 18.5$ ksi

$2.78F_y = (2.78)(33.0) = 91.7$ ksi

For $2.78F_y > F_e > 0.56F_y$:

$$F_c = \frac{10}{9}F_y\left(1 - \frac{10F_y}{36F_e}\right) \qquad (Eq.\ C3.1.2.1\text{-}2)$$

$$= \frac{10}{9}(33.0)\left(1 - \frac{(10)(33.0)}{(36)(58.6)}\right) = 30.93 \text{ ksi}$$

From Example I-9 with f = 30.93 ksi,

S_c = 0.606 in.^{3}

$M_n = S_cF_c$ (*Eq.* C3.1.2.1-1)

$= (0.606)(30.93)$

= 18.7 kip-in.

2. Permitted uniform live load, w_{live}

ASD

Allowable Strength (ASD)

$$M \le \frac{M_n}{\Omega_b} = \frac{18.7}{1.67} = 11.2 \text{ kip-in.}$$ (*Eq.* A4.1.1-1)

$$w_{live} \le \frac{8M}{L^2} = \frac{(8)(11.2)}{(72.0)^2} = 0.0173 \text{ kips/in.} = 207 \text{ plf}$$

LRFD

Design Strength (LRFD)

$$M_u \le \phi M_n = (0.90)(18.7) = 16.8 \text{ kip-in.}$$ (*Eq.* A5.1.1-1)

Live load factor = 1.6

$$1.6w_{live} \le \frac{(8)(16.8)}{(72.0)^2}$$

$$w_{live} \le 0.0162 \text{ kips/in.} = 194 \text{ plf}$$

3. Check Shear (Section C3.2.1)

$$h = D - 2(t+R) = 5.698 - 2(0.0566 + 0.0849) = 5.415 \text{ in.}$$

$$h/t = 5.415/0.0566 = 95.7$$

$$\sqrt{Ek_v/F_y} = \sqrt{(29500)(5.34)/33.0} = 69.1$$

$$1.51\sqrt{Ek_v/F_y} = (1.51)(69.1) = 104.3$$

For $\sqrt{Ek_v/F_y} < h/t \le 1.51\sqrt{Ek_v/F_y}$

$$F_v = \frac{0.6\sqrt{Ek_vF_y}}{(h/t)}$$ (*Eq.* C3.2.1-3)

$$= \frac{0.6\sqrt{(29500)(5.34)(33)}}{(95.7)}$$

= 14.3 ksi

$V_n = A_wF_v$ (*Eq.* C3.2.1-1)

$$= (5.415)(0.0566)(14.3) = 4.38 \text{ kips}$$

or per Table II-3, for a 550T125-54 with a yield stress of 33 ksi:

V_n = 4.38 kips

ASD

Allowable Strength

$$V = \frac{w_{live}L}{2} \leq \frac{V_n}{\Omega_v} \qquad (Eq.\ A4.1.1\text{-}1)$$

$$\Omega_v = 1.60$$

$$V = \frac{(0.0173)(72.0)}{2} = 0.62 \text{ kips} < \frac{4.38}{1.60} = 2.74 \text{ kips OK}$$

LRFD

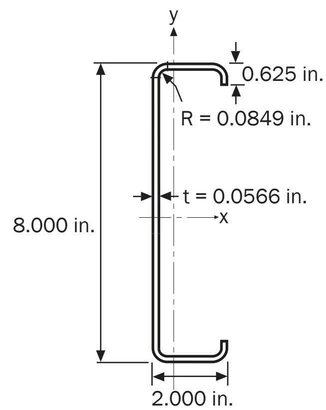
Design Strength

$$V_u = \frac{1.6w_{live}L}{2} < \phi_v V_n \qquad (Eq.\ A5.1.1\text{-}1)$$

$$\phi_v = 0.95$$

$$V_u = \frac{(1.6)(0.0162)(72.0)}{2} = 0.93 \text{ kips} < (0.95)(4.38) = 4.16 \text{ kips OK}$$

Example II-4: Distortional Buckling of C-Section



1. Steel: F_y = 50 ksi, F_u = 65 ksi
2. Section: 800S200-54 as shown above, spaced at 24 in. on center
3. The section is simply supported, uniformly loaded in flexure and fully braced against lateral-torsional buckling.
4. The section is sheeted on the compression flange with 7/16 in. OSB attached with #8 fasteners 12 in. on center. The rotational distortional buckling restraint provided by the sheathing is 0.0957 kip-in/rad./in.

Required:

Calculate the ASD flexural strength of the section, including consideration of distortional buckling.

Solution:

Gross section properties from Table I-2:

h_o = D = 8.000 in.
b_o = B = 2.000 in.
t = 0.0566 in.
d_o = d = 0.625 in.
R = 0.0849 in.
$S_f = S_x$ = 1.64 in.^{3}
x_o = –1.27 in.
θ = 90°

Effective section properties at initiation of yielding from Table II-2:

S_e = 1.50 in.^{3}

The allowable strength is the lowest calculated in accordance with sections C3.1.1 (nominal section strength), C3.1.2 (lateral-torsional buckling strength) and C3.1.4 (distortional buckling strength).

1. Nominal section strength – Section C3.1.1

Compute the strength at initiation of yielding, including the effects of local buckling, per Section 3.1.1(a).

$$M_n = S_e F_y \qquad (Eq.\ C3.1.1\text{-}1)$$

$$= (1.50)(50) = 75.0 \text{ kip-in. or } 6.25 \text{ kip-ft}$$

Calculate the allowable strength.

$$\frac{M_n}{\Omega_b} = \frac{6.25}{1.67} = 3.74 \text{ kip-ft} \qquad (\textit{Eq. } A4.1.1\text{-}1)$$

2. Lateral-torsional buckling strength – Section C3.1.2

Lateral-torsional buckling resistance is provided by the OSB. Requirements for the assumption of full lateral-torsional bracing of floor and roof systems are provided in Sections B1, B1.2.1 and B4 of the Floor and Roof System Design Standard (AISI-S210)[5]. Assume the OSB meets these requirements and provides full lateral-torsional bracing; therefore, the section is not subject to lateral-torsional buckling.

3. Distortional buckling strength – Section C3.1.4

Distortional buckling strength is determined using an elastic distortional buckling stress, F_d, which may be calculated by any of the three methods provided in Section C3.1.4. Each method will be considered separately below.

Calculate distortional buckling strength using the simplified approach in Section C3.1.4(a)

Section C3.1.4(a) provides a simplified and often very conservative estimate of the elastic distortional buckling stress, F_d. For sections meeting the geometric limitations, it can be used to produce a quick calculation to dismiss distortional buckling as a controlling limit state in some situations, but more accurate and liberal results can usually be obtained using Sections C3.1.4(b) and C3.1.4(c). This method does not account for the favorable restraining influence of the sheathing. The section 800S200-54 meets the geometric limitations of Section C3.1.4(a).

$$F_d = \beta k_d \frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{b_o}\right)^2 \qquad (\textit{Eq. } C3.1.4\text{-}6)$$

where

$$\beta = 1.0 \le 1 + 0.4(L/L_m)^{0.7}(1 - M_1/M_2)^{0.7} \le 1.3 \qquad (\textit{Eq. } C3.1.4\text{-}7)$$

β accounts for the favorable effect of a moment gradient. For the simply supported, uniformly loaded beam, the moments M_1 and M_2 on either side of the point of maximum moment are equal; therefore, β = 1.0 and there is no benefit.

$$k_d = 0.5 \le 0.6\left(\frac{b_o D \sin\theta}{h_o t}\right)^{0.7} \le 8.0 \qquad (\textit{Eq. } C3.1.4\text{-}9)$$

$$= 0.5 \le 0.6\left(\frac{2.000(0.625)(1.0)}{8.000(0.0566)}\right)^{0.7} \le 8.0$$

$$= 0.5 \le 1.22 \le 8.0 \quad \text{Use } 1.22$$

$$F_d = 1.0(1.22)\frac{\pi^2 29500}{12(1-0.3^2)}\left(\frac{0.0566}{2.000}\right)^2 = 26.1 \text{ ksi} \qquad (\textit{Eq. } C3.1.4\text{-}6)$$

Calculate the nominal distortional buckling moment per Section C3.1.4

[5] AISI S210, *North American Cold-Formed Steel Framing Standard*, *Floor and Roof System Design*, American Iron and Steel Institute, Washington, D.C.

$$M_y = S_{fy}F_y \qquad (Eq.\ C3.1.4\text{-}4)$$

$$= (1.64)(50) = 82.0\,\text{kip-in.}$$

$$M_{crd} = S_f F_d \qquad (Eq.\ C3.1.4\text{-}5)$$

$$= (1.64)(26.1) = 42.8\ \text{kip-in.}$$

$$\lambda_d = \sqrt{M_y/M_{crd}} \qquad (Eq.\ C3.1.4\text{-}3)$$

$$= \sqrt{82.0/42.8} = 1.38 > 0.673 \text{ therefore,}$$

$$M_n = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ C3.1.4\text{-}2)$$

$$= \left(1 - 0.22\left(\frac{42.8}{82.0}\right)^{0.5}\right)\left(\frac{42.8}{82.0}\right)^{0.5}(82.0) = 49.8\ \text{kip-in. or } 4.15\ \text{kip-ft}$$

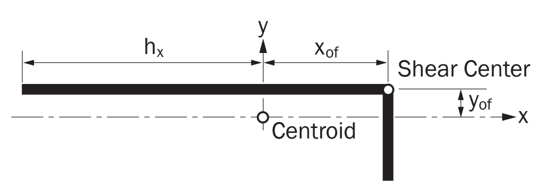
Calculate the allowable strength.

$$\frac{M_n}{\Omega_b} = \frac{4.15}{1.67} = 2.49\ \text{kip-ft} \qquad (Eq.\ A4.1.1\text{-}1)$$

Calculate distortional buckling strength using Section C3.1.4(b)

Section C3.1.4(b) provides a complex but more precise calculation method for the elastic distortional buckling stress, F_d. The formulae are based on analytical model of flexural-torsional buckling of the flange as a column restrained at the web/flange juncture by the available rotational stiffness from bending/buckling of the web plate. The *Specification* commentary provides formulae for the required cross-section properties in Table C-C3.1.4(b)-1. These formulae can also be found in *Manual* Part I, Section 3.4. Calculated properties for the tabulated standard cross sections are given in Tables II-7 through II-9.

Calculate flange properties for use in C3.1.4(b). Note that in the equations below, the *Specification* symbol x_o is presented as x_{of} to prevent confusion with similarly named symbol x_o used in Section C3.1.2. The symbol y_o is presented as y_{of} for consistency with x_{of}.



$$h = h_o - t$$

$$= 8.000 - 0.0566 = 7.943\ \text{in.}$$

$$b = b_o - t$$

$$= 2.000 - 0.0566 = 1.943\ \text{in.}$$

$$d = d_o - 0.5t$$

$$= 0.625 - 0.5(0.0566) = 0.597\ \text{in.}$$

$$A_f = (b + d)t$$

$$= (1.943 + 0.597)0.0566 = 0.144\ \text{in.}^2$$

$$I_{xf} = \frac{t\left(t^2b^2 + 4bd^3 + t^2bd + d^4\right)}{12\left(b+d\right)}$$

$$= \frac{0.0566\begin{pmatrix}(0.0566)^2(1.943)^2 + 4(1.943)(0.597)^3 \\ +(0.0566)^2(1.943)(0.597) + (0.597)^4\end{pmatrix}}{12(1.943+0.597)} = 0.00334 \text{ in.}^4$$

$$I_{yf} = \frac{t\left(b^4 + 4db^3\right)}{12\left(b+d\right)}$$

$$= \frac{0.0566\left((1.943)^4 + 4(0.597)(1.943)^3\right)}{12(1.943+0.597)} = 0.0590 \text{ in.}^4$$

$$I_{xyf} = \frac{tb^2d^2}{4\left(b+d\right)}$$

$$= \frac{0.0566(1.943)^2(0.597)^2}{4(1.943+0.597)} = 0.00750 \text{ in.}^4$$

$$x_{of} = \frac{b^2}{2\left(b+d\right)}$$

$$= \frac{(1.943)^2}{2(1.943+0.597)} = 0.743 \text{ in.}$$

$$y_{of} = \frac{-d^2}{2\left(b+d\right)}$$

$$= \frac{-(0.597)^2}{2(1.943+0.597)} = -0.0702 \text{ in.}$$

$$h_x = \frac{-\left(b^2 + 2db\right)}{2\left(b+d\right)}$$

$$= \frac{-\left((1.943)^2 + 2(0.597)(1.943)\right)}{2(1.943+0.597)} = -1.20 \text{ in.}$$

$$J_f = \frac{bt^3 + dt^3}{3} = \frac{(1.943)(0.0566)^3 + (0.597)(0.0566)^3}{3} = 0.000154 \text{ in.}^4$$

$$C_{wf} = 0.0 \text{ in.}^6$$

Calculate the length, L_{cr}, over which the elastic distortional buckling half-wave forms if there is no distortional bucking restraint from the sheathing.

$$L_{cr} = \left(\frac{4\pi^4 h_o\left(1-\mu^2\right)}{t^3}\left(I_{xf}\left(x_{of}-h_x\right)^2 + C_{wf} - \frac{I_{xyf}^2}{I_{yf}}\left(x_{of}-h_x\right)^2\right) + \frac{\pi^4 h_o^4}{720}\right)^{1/4} \qquad (Eq.\ C3.1.4\text{-}12)$$

where x_o is replaced by x_{of} as discussed above.

$$L_{cr} = \left(\frac{4\pi^4 (8.000)\left(1-0.3^2\right)}{(0.0566)^3} \left(\begin{array}{l} 0.00334\left(0.743-(-1.20)\right)^2 + 0.0 \\ -\dfrac{(0.00750)^2}{0.0590}\left(0.743-(-1.20)\right)^2 \end{array} \right) + \frac{\pi^4 (8.000)^4}{720} \right)^{1/4} = 19.4 \text{ in.}$$

Based on the assumption of continuous partially effective bracing for distortional buckling, take the assumed distortional buckling length, L, as equal to L_{cr}.

$L = L_{cr} = 19.4$ in.

Calculate the elastic rotational stiffness provided by the flange to the flange/web junction, $k_{\phi fe}$.

$$k_{\phi fe} = \left(\frac{\pi}{L}\right)^4 \left(EI_{xf}\left(x_{of} - h_x\right)^2 + EC_{wf} - E\frac{I_{xyf}^2}{I_{yf}}\left(x_{of} - h_x\right)^2 \right) + \left(\frac{\pi}{L}\right)^2 GJ_f \qquad (Eq.\ C3.1.4\text{-}13)$$

where x_o is replaced by x_{of} as discussed above.

$$k_{\phi fe} = \left(\frac{\pi}{19.4}\right)^4 \left(\begin{array}{l} (29500)(0.00334)\left(0.743-(-1.20)\right)^2 \\ +(29500)(0.0) \\ -29500\dfrac{(0.00750)^2}{0.0590}\left(0.743-(-1.20)\right)^2 \end{array} \right) + \left(\frac{\pi}{19.4}\right)^2 (11300)(0.000154)$$

= 0.228 kips

Calculate the elastic rotational stiffness provided by the web to the flange/web junction, $k_{\phi we}$.

$$k_{\phi we} = \frac{Et^3}{12\left(1-\mu^2\right)} \left(\frac{3}{h_o} + \left(\frac{\pi}{L}\right)^2 \frac{19h_o}{60} + \left(\frac{\pi}{L}\right)^4 \frac{h_o^3}{240} \right) \qquad (Eq.\ C3.1.4\text{-}14)$$

$$= \frac{29500(0.0566)^3}{12\left(1-(0.3)^2\right)} \left(\frac{3}{8.000} + \left(\frac{\pi}{19.4}\right)^2 \frac{19(8.000)}{60} + \left(\frac{\pi}{19.4}\right)^4 \frac{(8.000)^3}{240} \right)$$

= 0.217 kips

Calculate the geometric rotational stiffness demanded by the flange from the flange/web junction, $\tilde{k}_{\phi fg}$.

$$\tilde{k}_{\phi fg} = \left(\frac{\pi}{L}\right)^2 \left[A_f \left(\left(x_{of} - h_x\right)^2 \left(\frac{I_{xyf}}{I_{yf}}\right)^2 - 2y_{of}\left(x_{of} - h_x\right)\left(\frac{I_{xyf}}{I_{yf}}\right) + h_x^2 + y_{of}^2 \right) + I_{xf} + I_{yf} \right]$$

(*Eq.* C3.1.4-15)

Using x_{of} for x_o and y_{of} for y_o as discussed above,

$$\tilde{k}_{\phi fg} = \left(\frac{\pi}{19.4}\right)^2 \left[0.144\left(\begin{array}{l}(0.743-(-1.20))^2\left(\dfrac{0.00750}{0.0590}\right)^2 \\ -2(-0.0702)(0.743-(-1.20))\left(\dfrac{0.00750}{0.0590}\right) \\ +(-1.20)^2+(-0.0702)^2\end{array}\right)+0.00334+0.0590\right]$$

$= 0.00745 \text{ in.}^2$

Calculate the geometric rotational stiffness demanded by the web from the flange/web junction, $\tilde{k}_{\phi wg}$.

$$\tilde{k}_{\phi wg} = \frac{h_o t\pi^2}{13440}\left(\frac{\left[45360(1-\xi_{web})+62160\right]\left(\dfrac{L}{h_o}\right)^2+448\pi^2+\left(\dfrac{h_o}{L}\right)^2\left[53+3(1-\xi_{web})\right]\pi^4}{\pi^4+28\pi^2\left(\dfrac{L}{h_o}\right)^2+420\left(\dfrac{L}{h_o}\right)^4}\right)$$

(*Eq.* C3.1.4-16)

$$= \frac{8.000(0.0566)\pi^2}{13440}\left(\frac{\left[45360(1-2)+62160\right]\left(\dfrac{19.4}{8.000}\right)^2 + 448\pi^2+\left(\dfrac{8.000}{19.4}\right)^2\left[53+3(1-2)\right]\pi^4}{\pi^4+28\pi^2\left(\dfrac{19.4}{8.000}\right)^2+420\left(\dfrac{19.4}{8.000}\right)^4}\right)$$

$= 0.00213 \text{ in}^2$.

In *Eq.* C3.1.4-16 above, ξ_{web} is taken as 2, since the section is in pure symmetrical bending.

Alternatively, for this standard section, the values of $k_{\phi fe}, k_{\phi we}, \tilde{k}_{\phi fg}$ and $\tilde{k}_{wg}$ could be taken from Table II-8 as 0.229 kips, 0.217 kips, 0.00745 in.^{2} and 0.00213 in.^{2}, respectively.

Calculate the elastic distortional buckling stress, F_d.

$$F_d = \beta\frac{k_{\phi fe}+k_{\phi we}+k_\phi}{\tilde{k}_{fg}+\tilde{k}_{wg}} \qquad (Eq.\ C3.1.4\text{-}10)$$

Since there is no beneficial moment gradient at the midspan, $\beta = 1.0$.

If the beneficial restraining effect of the floor sheathing is ignored for simplicity, $k_\phi = 0.0$.

$$F_d = (1.0)\frac{0.228+0.217+0.0}{0.00745+0.00213} = 46.5 \text{ ksi} \qquad (Eq.\ C3.1.4\text{-}10)$$

Alternatively, for this standard section, the value of F_d/β could be taken from Table II-8 as 46.5 ksi.

Note that this elastic distortional buckling stress is approximately 80% higher than that calculated using the simplified provisions of Section C3.1.4(a)

If the restraining effect of the floor sheathing is included, k_ϕ = 0.0957 kips

$$F_d = (1.0)\frac{0.228+0.217+0.0957}{0.00745+0.00213} = 56.4 \text{ ksi} \qquad (Eq.\ C3.1.4\text{-}10)$$

Note that this elastic distortional buckling stress is approximately 120% higher than that calculated using the simplified provisions of Section C3.1.4(a)

Calculate the nominal distortional buckling moment per Section C3.1.4, considering the sheathing.

$$M_y = 82.0 \text{ kip-in. (from above)}$$

$$M_{crd} = S_f F_d \qquad (Eq.\ C3.1.4\text{-}5)$$

$$= (1.64)(56.4) = 92.5 \text{ kip-in.}$$

$$\lambda_d = \sqrt{M_y/M_{crd}} \qquad (Eq.\ C3.1.4\text{-}3)$$

$$= \sqrt{82.0/92.5} = 0.942 > 0.673 \text{ therefore,}$$

$$M_n = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ C3.1.4\text{-}2)$$

$$= (1-0.22(1/0.942))(1/0.942)(82.0) = 66.7 \text{ kip-in. or } 5.56 \text{ kip-ft}$$

Calculate the allowable distortional buckling strength.

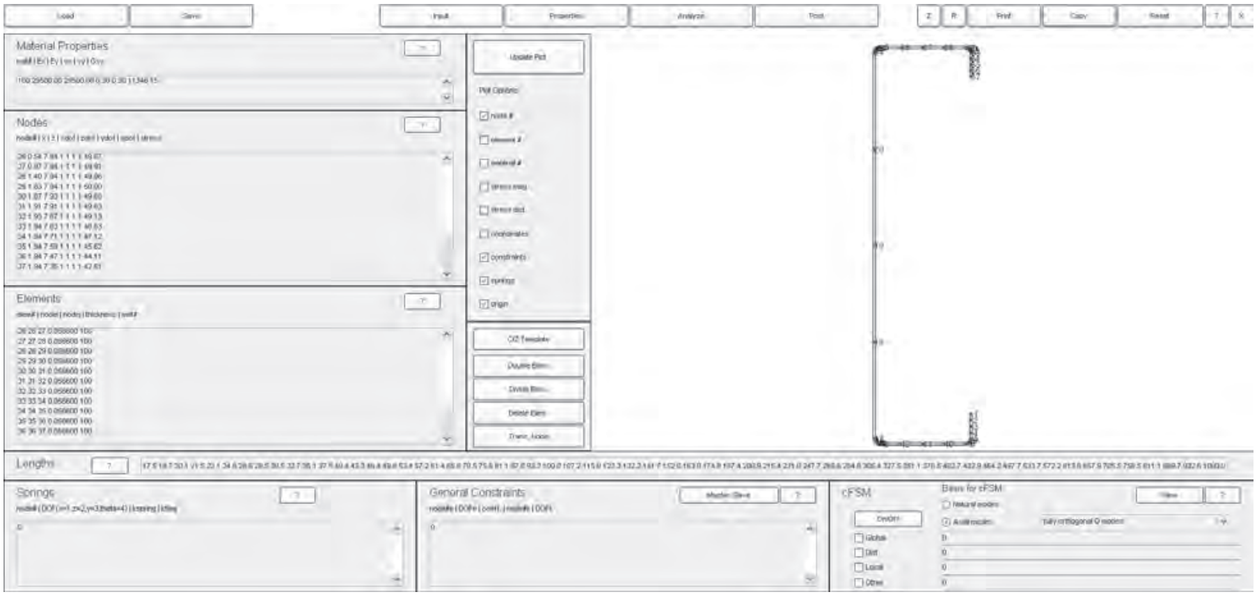
$$\frac{M_n}{\Omega_b} = \frac{5.56}{1.67} = 3.33 \text{ kip-ft} \qquad (Eq.\ A4.1.1\text{-}1)$$

Calculate distortional buckling strength using Section C3.1.4(c)

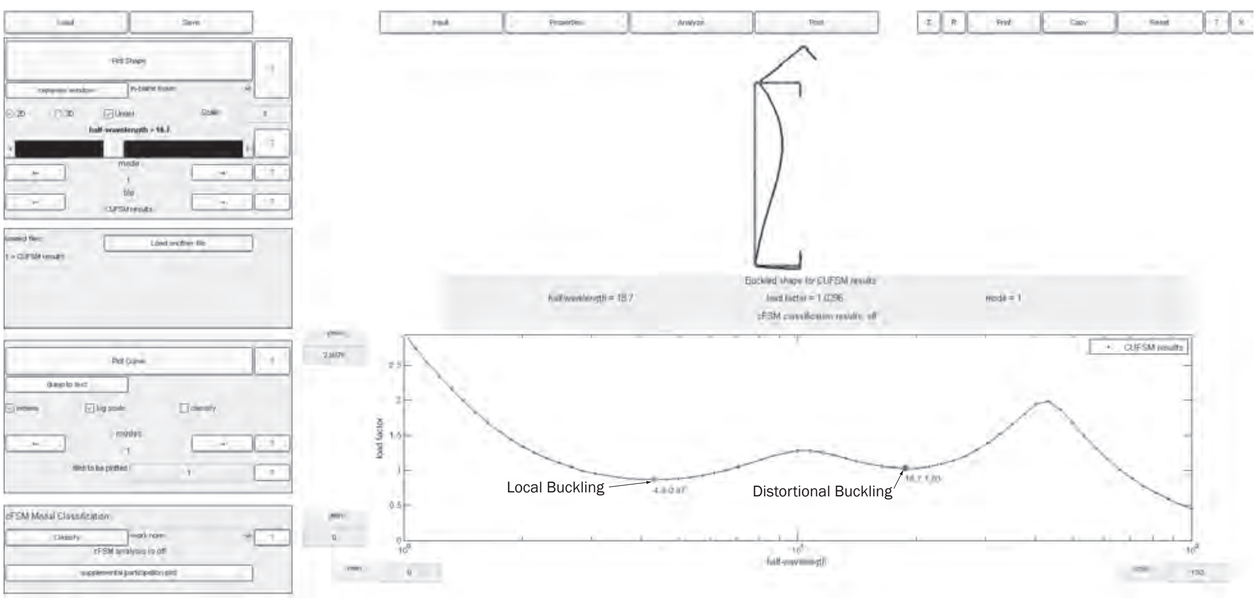
Section C3.1.4(c) permits the use of a "rational buckling analysis" to determine the elastic distortional buckling strength stress, F_d. The approach followed below determines the elastic distortional buckling stress, F_d, from a elastic buckling computer model. The stress, F_d, is then used in the calculation of the distortional buckling strength in a manner similar to the calculations above.

The cross section is modeled using the finite strip software program CUFSM[6]. The input screen is shown below.

[6] Schafer, B.W., Ádány, S. "Buckling analysis of cold-formed steel members using CUFSM: conventional and constrained finite strip methods." Eighteenth International Specialty Conference on Cold-Formed Steel Structures, Orlando, FL. October 2006.



The model is built using the program's C/Z template. No rotational restraint of the compression flange, k_ϕ, is included in this version of the model. A linear stress gradient from F_y in compression at the top to F_y in tension at the bottom is applied as a reference loading. The results of the analysis are shown on the screen below.

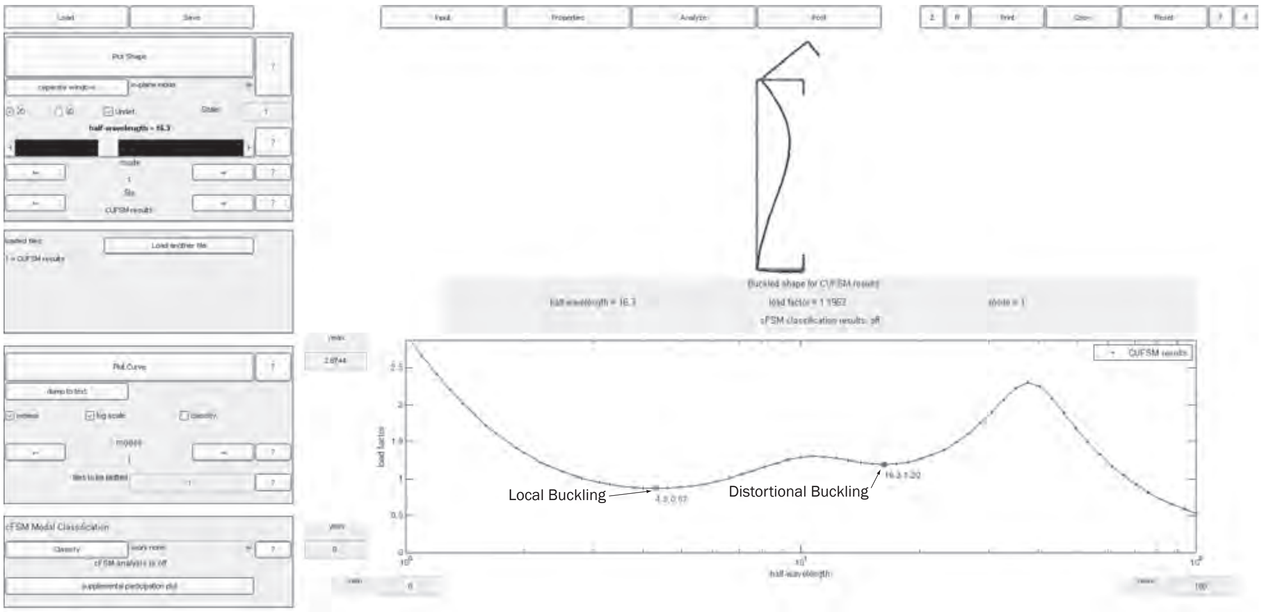


The result plots show that, without rotational restraint of the compression flange, distortional buckling occurs at a half-wavelength of 18.7 in. at a load factor of 1.03 times the applied loading. Thus, the elastic distortional buckling stress, F_d is computed as,

$$F_d = 1.03F_y = (1.03)(50.0) = 51.5 \text{ ksi}$$

This can be compared with the elastic distortional buckling stress of 26.1 ksi calculated using Section C3.1.4(a) and 46.5 ksi calculated using Section C3.1.4(b).

When a rotational stiffness of 0.0957 kip-in./rad./in. from the sheathing is added to the joint at the center of the compression flange, the results of the analysis are as shown in the following figure.



With rotational restraint of the compression flange, distortional buckling occurs at a loading of 1.20 times the applied load. Thus, the elastic distortional buckling stress, F_d is computed as,

$$F_d = 1.20F_y = (1.20)(50.0) = 60.0 \text{ ksi}$$

This can be compared to an elastic distortional buckling stress of 56.4 ksi calculated from Section 3.1.4(b)

Calculate the nominal distortional buckling moment per Section C3.1.4.

$$M_y = = 82.0 \text{ kip-in. (from above)}$$

$$M_{crd} = S_f F_d \qquad (Eq.\ C3.1.4\text{-}5)$$

$$= (1.64)(60.0) = 98.4 \text{ kip-in.}$$

$$\lambda_d = \sqrt{M_y / M_{crd}} \qquad (Eq.\ C3.1.4\text{-}3)$$

$$= \sqrt{82.0/98.4} = 0.913 > 0.673 \text{ therefore,}$$

$$M_n = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ C3.1.4\text{-}2)$$

$$= (1 - 0.22(1/0.913))(1/0.913)(82.0) = 68.2 \text{ kip-in. or } 5.68 \text{ kip-ft}$$

Calculate the allowable distortional buckling strength.

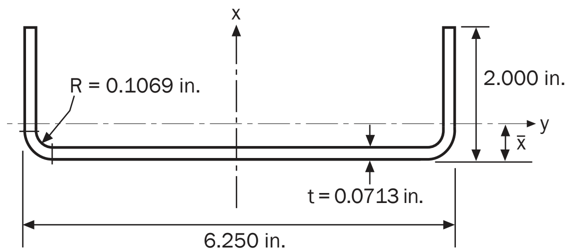
$$\frac{M_n}{\Omega_b} = \frac{5.68}{1.67} = 3.40 \text{ kip-ft} \qquad (Eq.\ A4.1.1\text{-}1)$$

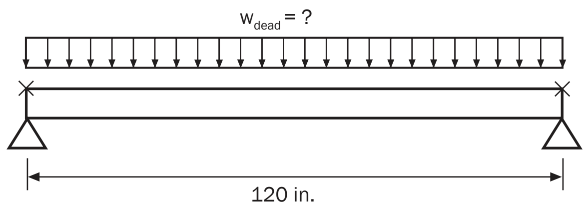
The table below compares the results of the three alternative approaches to calculating the distortional buckling strength with the strength calculated per Section C3.1.1 at the initiation of yielding, including the effects of local buckling.

Limit State	Specification Section	Allowable Flexural Strength M_n/Ω_b , kip-ft	
		Not Considering Sheathing Rotational Restraint	Considering Sheathing Rotational Restraint
Distortional Buckling	C3.1.4(a)	2.49	-
	C3.1.4(b)	-	3.33
	C3.1.4(c)	-	3.40
Yielding, including local buckling	C3.1.1(a)	3.74	

Note that Sections C3.1.4(b) and C3.1.4(c) produce similar results, both of which are significantly higher than those from C3.1.4(a). Since the distortional buckling strength is lower than the strength based on initiation of yielding, distortional buckling governs, regardless of the method selected from Section C3.1.4 to evaluate distortional buckling.

Example II-5: C-Section Without Lips in Weak Axis Bending





t = 0.0713 in.
R = 0.1069 in.
A = 0.712 in.[2]
I_x = 3.99 in.[4]
r_x = 2.37 in.
$\bar{x}$ = 0.422 in.[2]
S_y = 0.161 in.[3]
J = 0.00121 in.[4]
C_w = 1.75 in.[6]
j = 3.48 in.
r_o = 2.65 in.

Given:

1. Steel: F_y = 33 ksi
2. Section: SSMA Track 600T200-68 oriented with the lips up as shown in sketch above, used as a cable tray supporting dead load only.
3. Simple span of 120 in.
4. Braced against twisting and lateral deflection at ends. Ends reinforced against crippling

Required:

1. Largest permitted uniformly distributed service dead load, w_{dead}, using ASD and LRFD. Compute w_{dead} based on flexural strength, ignoring shear and serviceability.

Solution:

The beam is subject to lateral-torsional buckling, but not subject to distortional buckling.

1. Lateral-torsional buckling strength (Section C3.1.2.1)

For singly-symmetric sections bent about the centroidal axis perpendicular to the axis of symmetry, use Section C3.1.2.1(a)(ii)

$$F_e = \frac{C_s A \sigma_{ex}}{C_{TF} S_f}\left[j + C_s\sqrt{j^2 + r_o^2(\sigma_t/\sigma_{ex})}\right] \qquad (Eq.\ C3.1.2.1\text{-}10)$$

$$\sigma_{ex} = \frac{\pi^2 E}{(K_x L_x/r_x)^2} \qquad (Eq.\ C3.1.2.1\text{-}11)$$

$$= \frac{\pi^2(29500)}{((1.0)(120)/2.37)^2}$$

$$= 113.6 \text{ ksi}$$

$$\sigma_t = \frac{1}{A r_o^2}\left[GJ + \frac{\pi^2 E C_w}{(K_t L_t)^2}\right] \qquad (Eq.\ C3.1.2.1\text{-}9)$$

$$\sigma_t = \frac{1}{(0.712)(2.65)^2}\left[(11300)(0.00121)+\frac{\pi^2(29500)(1.75)}{\left[(1.0)(120)\right]^2}\right]$$

$= 9.81$ ksi

C_s = −1, since there is tension on the shear center side of the centroid

C_{TF} = 1, since the midspan moment is larger than the end moments

$$F_e = \frac{(-1)(0.712)(113.6)}{(1)(0.161)}\left[3.48+(-1)\sqrt{(3.48)^2+(2.65)^2((9.81)/(113.6))}\right] \quad (Eq.\ C3.1.2.1\text{-}10)$$

= 43.24 ksi

$0.56F_y$ = (0.56)(33.0) = 18.5 ksi

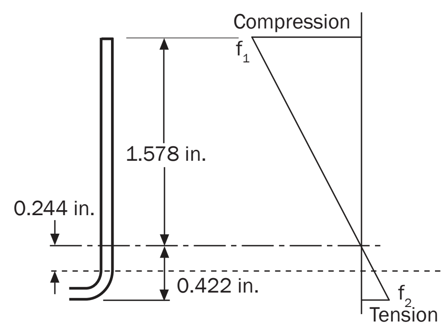
$2.78F_y$ = (2.78)(33.0) = 91.7 ksi

For $2.78F_y > F_e > 0.56F_y$:

$$F_c = \frac{10}{9}F_y\left(1-\frac{10F_y}{36F_e}\right) \quad (Eq.\ C3.1.2.1\text{-}2)$$

$$= \frac{10}{9}(33.0)\left(1-\frac{(10)(33.0)}{(36)(43.24)}\right) = 28.89 \text{ ksi}$$

2. Calculate the effective section modulus, S_y, at F_c = 28.89 ksi



The horizontal channel web is in tension; therefore it is fully effective. Evaluate the vertical flanges under the stress gradient.

f_1 = 28.89 ksi, maximum at tips of flanges

Assuming the section is fully effective for the first approximation,

$$f_2 = -28.89\frac{0.422-0.0713-0.1069}{2.000-0.422} = -4.46 \text{ ksi}$$

Using Section B3.2(a)(2)

$$\psi = |f_2/f_1| = |-4.46/28.89| = 0.154 \quad (Eq.\ B3.2\text{-}1)$$

$$k = 0.57+0.21\psi+0.07\psi^2 \quad (Eq.\ B3.2\text{-}5)$$

$$= 0.57+0.21(0.154)+0.07(0.154)^2 = 0.604$$

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \quad (Eq.\ B2.1\text{-}5)$$

$$= 0.604\frac{\pi^2 29500}{12(1-0.3^2)}\left(\frac{0.0713}{1.822}\right)^2 = 24.66 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \quad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{28.89}{24.66}} = 1.082$$

$0.673(1+\psi) = 0.673(1+0.154) = 0.777 < \lambda$; therefore,

$$\rho = (1+\psi)\frac{\left(1-\frac{0.22(1+\psi)}{\lambda}\right)}{\lambda} \qquad (Eq.\ B3.2\text{-}4)$$

$$= (1+0.154)\frac{\left(1-\frac{0.22(1+0.154)}{1.082}\right)}{1.082} = 0.816$$

$$b = \rho w \qquad (Eq.\ B2.1\text{-}2)$$

$$= 0.816(1.822) = 1.487 \text{ in.}$$

Using Alternative 1 method for unstiffened C-sections

Check applicability

$b_o/h_o = 2.000/6.250 = 0.320$

$0.1 \le 0.320 \le 1.0$ OK

$$k = 0.145(b_o/h_o) + 1.256 \qquad (Eq.\ B3.2\text{-}11)$$

$$= 0.145(2.000/6.250) + 1.256 = 1.302$$

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 1.302\frac{\pi^2 29500}{12(1-0.3^2)}\left(\frac{0.0713}{1.822}\right)^2 = 53.16 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$= \sqrt{\frac{28.89}{53.16}} = 0.737 < 0.856$; therefore the flange is fully effective

$$b = w = 1.822 \text{ in.} \qquad (Eq.\ B2.1\text{-}2)$$

3. Calculate the nominal strength, M_n

Use the results of the less conservative Alternative 1 method. All elements are fully effective; therefore, use the full section modulus.

$S_c = S_y = 0.161$ in.^{3}

$$M_n = S_c F_c \qquad (Eq.\ C3.1.2.1\text{-}1)$$

$$= (0.161)(28.89)$$

$$= 4.65 \text{ kip-in.}$$

4. Permitted uniform service dead load, w_{dead}

ASD

Allowable Strength

$$M \le \frac{M_n}{\Omega_b} = \frac{4.65}{1.67} = 2.78 \text{ kip-in.} \qquad (Eq.\ A4.1.1\text{-}1)$$

$$w_{dead} \le \frac{8M}{L^2} = \frac{(8)(2.78)}{(120)^2} = 0.00154 \text{ kips/in.} = 18.5 \text{ plf}$$

LRFD

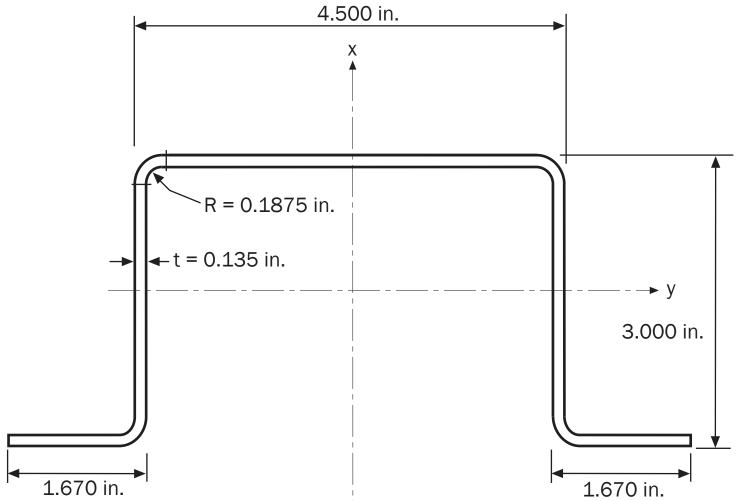
Design Strength

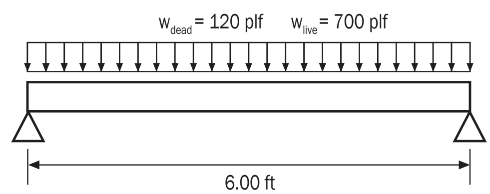
$M_u \leq \phi M_n = (0.90)(4.65) = 4.19$ kip-in. (*Eq.* A5.1.1-1)

Dead load factor = 1.4

$$w_{dead} \leq \frac{(8)(4.19)}{(120)^2(1.4)} = 0.00166 \text{ kips/in.} = 20.0 \text{ plf}$$

Example II-6: Fully Braced Hat Section





Given:

1. Steel: F_y = 50 ksi
2. Section: 3HU4.5x135 shown in sketch above
3. Top flange is in compression and is fully braced.

Required:

1. Check the flexural adequacy of a 6 foot long simple span beam with:
   Dead Load, w_{dead} = 120 plf
   Live Load, w_{live} = 700 plf
2. Do not consider inelastic reserve.
3. Check using both ASD and LRFD.

Solution:

1. Calculate Nominal Strength, M_n (Section C3.1.1)

Since the member is fully braced and not subject to lateral-torsional or distortional buckling, calculate the nominal strength using Section C3.1.1.

From Example I-13 or Table II-6, S_e = 1.52 in.^{3}

$$M_n = S_eF_y \qquad (Eq.\ C3.1.1\text{-}1)$$

$$= (1.52)(50) = 76.0\ \text{kip-in.}$$

2. Calculate Required and Available Strength

Bending Moments

$$M_D = \frac{w_{dead}L^2}{8} = \frac{(0.120)(6)^2}{8} = 0.540\ \text{kip-ft} = 6.48\ \text{kip-in.}$$

$$M_L = \frac{w_{live}L^2}{8} = \frac{(0.700)(6)^2}{8} = 3.15\ \text{kip-ft} = 37.8\ \text{kip-in.}$$

ASD

Required Allowable Strength

$M = M_D + M_L = 6.48 + 37.8 = 44.3$ kip-in.

Allowable Strength

$M < M_n/\Omega_b$ (*Eq.* A4.1.1-1)

$\Omega_b = 1.67$

$$\frac{M_n}{\Omega_b} = \frac{76.0}{1.67} = 45.5 \text{ kip-in.} > 44.3 \text{ kip-in. OK}$$

LRFD

Required Strength

$M_u = 1.2M_D + 1.6\ M_L$

$= (1.2)(6.48) + (1.6)(37.8)$

$= 68.3$ kip-in.

Design Strength

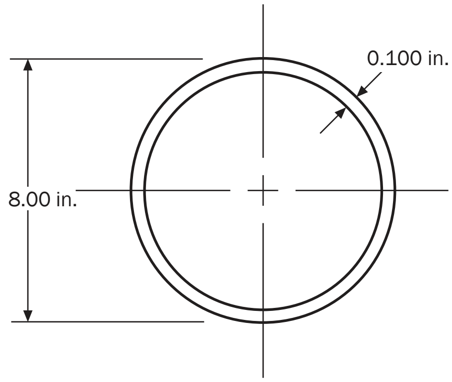
$M_u < \phi_b M_n$ (*Eq.* A5.1.1-1)

$\phi_b = 0.95$

$\phi_b M_n = (0.95)(76.0) = 72.2$ kip-in. > 68.3 kip-in. OK

The resistance factor of 0.95 is permitted per Section C3.1.1 because the compression flange is stiffened.

Example II-7: Tubular Section – Round



Given:

1. Steel: F_y = 42 ksi
2. Section: Shown in sketch above

Required:

1. Determine the ASD allowable flexural strength, M_n/Ω_b .
2. Determine the LRFD design flexural strength, $\phi_b M_n$.

Solution:

1. Nominal Flexural Strength (Section C3.1.3):

Ratio of outside diameter to wall thickness,

$D/t = 8.00/0.100 = 80.0$

Check limit

$D/t < 0.441E/F_y = 0.441(29500/42) = 310$ OK

Full Section Properties

$$S_f = \pi\frac{(\text{Outside Diameter})^4 - (\text{Inside Diameter})^4}{32(\text{Outside Diameter})}$$

$$= \pi\frac{(8.00)^4 - (7.80)^4}{(32)(8.00)}$$

$= 4.84$ in.^{3}

Determine the governing equation

$0.0714E/F_y = 0.0714(29500/42) = 50.2$

$0.318E/F_y = 0.318(29500/42) = 223$

Since $0.0714E/F_y < D/t < 0.318E/F_y$

$$F_c = \left[0.970 + 0.020\left(\frac{E/F_y}{D/t}\right)\right]F_y \qquad (Eq.\ C3.1.3\text{-}3)$$

$$F_c = \left[0.970 + 0.020\left(\frac{29500/42}{80.0}\right)\right]42 = 48.1 \text{ ksi}$$

$$M_n = F_c S_f \quad (Eq.\ C3.1.3\text{-}1)$$

$$M_n = (48.1)(4.84) = 233 \text{ kip-in.}$$

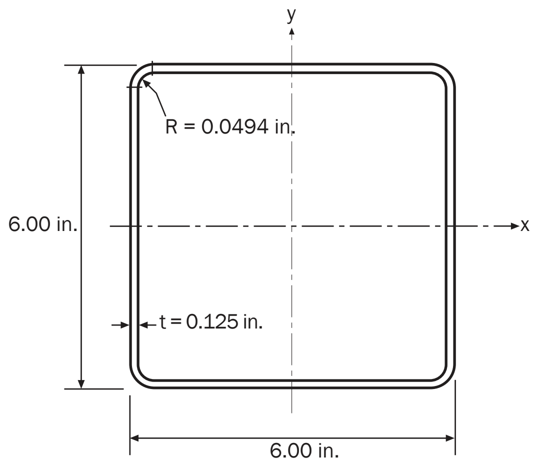
2. ASD allowable flexural strength

$$\frac{M_n}{\Omega_b} = \frac{233}{1.67} = 140 \text{ kip-in.}$$

3. LRFD design flexural strength

$$\phi_b M_n = (0.95)(233) = 221 \text{ kip-in.}$$

Example II-8: Tubular Section – Rectangular



Given:

1. Steel: F_y = 46 ksi
2. Section: HSS 6x6x⅛ (from Table 1-12, AISC *Steel Construction Manual*, 2005)
3. Calculated gross properties using nominal dimensions above
   A = 2.91 in.^{2}
   I = 16.7 in.^{4}
   S = 5.56 in.^{3}
3. Simple span length = 10.0 ft
4. Laterally braced at both ends

Required:

1. Determine the ASD flexural allowable strength, M_n/Ω_b .
2. Determine the LRFD design flexural strength, $\phi_b M_n$.
3. Compare the calculated available flexural strengths to those calculated using the AISC procedures. The inside radius given above is selected to give the same flat width, w, for the flanges and webs used in the AISC calculations.

Solution:

Compute the nominal flexural strength, M_n, according to *Specification* Sections C3.1.1(a) and C3.1.2.2 for closed box members.

1. Nominal Flexural Strength, M_n, based on initiation of yielding (Section C3.1.1):

Check the compression flange as a uniformly compressed compression element in accordance with Section B2.1. By calculations not shown, the effective width of the compression flange width is found to be 4.608 in.

Check webs in accordance with Section B2.3. The webs are found to be fully effective, by calculations not shown.

The effective section properties can then be calculated as:

I_x = 15.5 in.^{4}

S_e = 4.95 in.^{3}

The nominal flexural strength is calculated as:

$$M_n = S_e F_y \qquad (Eq.\ C3.1.1\text{-}1)$$

$$= (4.95)(46) = 228 \text{ kip-in. or } 19.0 \text{ kip-ft}$$

2. Nominal Flexural Strength, M_n, based on lateral-torsional buckling (Section C3.1.2.2):

According to *Specification Eq.* C3.1.2.2-1, the minimum unbraced length of a closed box member subject to lateral-torsional buckling, L_u, is calculated as:

$$L_u = \frac{0.36 C_b \pi}{F_y S_f}\sqrt{EGJI_y} \qquad (Eq.\ C3.1.2.2\text{-}1)$$

where

C_b = 1.0 (assumed)

I_y (full section) = 16.7 in.^{4}

S_f (full section) = 5.56 in.^{3}

$$J = \frac{2(ab)^2}{(a/t_1) + (b/t_2)} \qquad (Eq.\ C\text{-}C3.1.2.2\text{-}1)$$

where

$a = b = D - t = 6.00 - 0.125 = 5.875$ in.

$t_1 = t_2 = 0.125$ in.

$$J = \frac{2(5.875)^4}{2(5.875/0.125)} = 25.3 \text{ in.}^4 \qquad (Eq.\ C\text{-}C3.1.2.2\text{-}1)$$

therefore,

$$L_u = \frac{0.36(1.0)\pi}{(46)(5.56)}\sqrt{(29500)(11300)(25.3)(16.7)} = 1660 \text{ in.} = 138 \text{ ft} \qquad (Eq.\ C3.1.2.2\text{-}1)$$

Since L_u > 10.0 ft, the member is not subject to lateral-torsional buckling and the flexural strength is based on Section C3.1.1(a). That is,

M_n = 19.0 kip-ft

3. ASD allowable flexural strength

$$\frac{M_n}{\Omega_b} = \frac{19.0}{1.67} = 11.4 \text{ kip-ft}$$

4. LRFD design flexural strength

$$\phi_b M_n = (0.95)(19.0) = 18.1 \text{ kip-ft.}$$

5. Comparison of AISI and AISC available flexural strengths

The available flexural strengths calculated above are compared with those calculated in accordance with the 2005 AISC Specification in the table below. AISC strengths are taken from Table 3-13 of the 13th Edition of the AISC Steel Construction Manual.

Available Strength	AISI kip-ft	AISC kip-ft	$\frac{\text{AISI}}{\text{AISC}}$
ASD: M_n/Ω_b	11.4	10.8	1.06
LRFD: $\phi_b M_n$	18.1	16.2	1.12

The differences between the AISI and AISC strengths shown in the table above are due to the following factors:

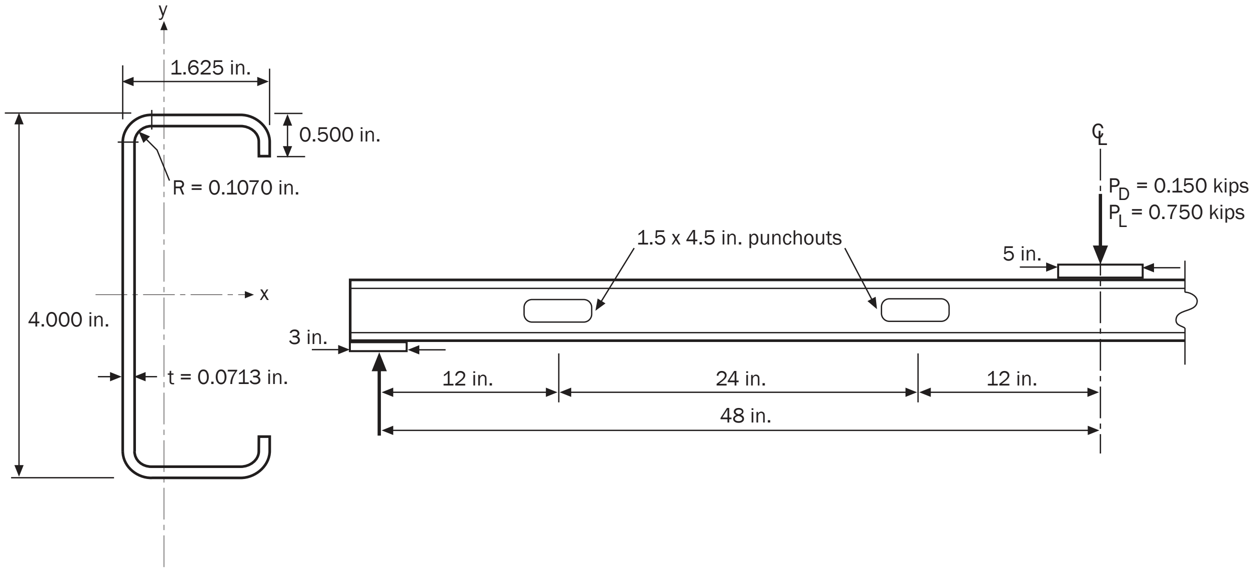
a. The AISI calculations were performed using the nominal wall thickness of the HSS, while the AISC calculations were performed using 93% of the nominal wall thickness as required by AISC.

b. The calculations of the effective width of the compression flange are slightly different in the two specifications, including slightly different values of E.

c. For the LRFD method, the resistance factor used in the AISI *Specification* is $\phi_b = 0.95$ while the resistance factor used in the AISC Specification is $\phi_b = 0.90$.

d. The differences shown in the table above are representative of HSS sections having slender elements, but actual ratios vary somewhat based on section dimensions and yield stress.

Example II-9: C-Section with Openings



Given:

1. Steel: F_y = 50 ksi, F_u = 65 ksi
2. Section: 400S162-68 as shown above
3. Section is simply supported, fully braced against lateral-torsional and distortional buckling, and is fastened to support.
4. 1.5 in. by 4.5 in. web punchouts with 0.25 inch corner radii located as shown above. Note that the location of punchouts is often not known with this precision.

Required:

Check the adequacy of the section considering:

a. Flexure
b. Shear
c. Web Crippling
d. Combined Bending and Shear
e. Combined Bending and Web Crippling

Use:

1. ASD – ASCE/SEI 7-05 ASD load combination D + L
2. LRFD - ASCE/SEI 7-05 LRFD load combination 1.2D + 1.6L

Neglect self weight of beam

Solution:

1. Flexural Strength

a) Required Strength

ASD Required Strength

$$P = P_D + P_L = 0.150 + 0.750 = 0.900 \text{ kips}$$

$$V = P/2 = 0.900/2 = 0.450 \text{ kips}$$

At center, away from holes

$$M = \frac{PL}{4} = \frac{(0.900)(8.0)}{4} = 1.80 \text{ kip-ft} = 21.6 \text{ kip-in.}$$

At edge of hole closest to center

$$M = V\left[L/2-(12.0-2.25)\right]=(0.450)\left[96.0/2-9.75\right]=17.2 \text{ kip-in.}$$

LRFD Required Strength

$$P_u = 1.2P_D + 1.6P_L = (1.2)(0.150)+(1.6)(0.750)=1.38 \text{ kips}$$

$$V_u = P_u/2 = 1.38/2 = 0.690 \text{ kips}$$

At center, away from holes

$$M_u = \frac{P_uL}{4} = \frac{(1.38)(8.0)}{4} = 2.76 \text{ kip-ft} = 33.1 \text{ kip-in.}$$

At edge of hole closest to center

$$M_u = V_u\left[L/2-(12.0-2.25)\right]=(0.690)\left[(96.0/2)-9.75\right]=26.4 \text{ kip-in.}$$

b) Flexural Strength without Holes

The member is not subject to lateral-torsional buckling, so compute strength using Section C3.1.1 with effective section modulus, S_e, at $f = F_y$.

It can be shown that, in the area without holes, the section is eligible for strength increase using the cold work of forming provisions of Section A7.2.

$F_y = F_{ya} = 56.6$ ksi (calculations not shown)

$S_e = 0.670$ in.^{3} (calculations not shown)

$M_n = S_eF_y$ (*Eq.* C3.1.1-1)

$= (0.670)(56.6) = 37.9$ kip-in.

c) Nominal Flexural Strength with Holes

The member is not subject to lateral-torsional buckling, so compute strength using Section C3.1.1 with effective section modulus, S_e, at $f = F_y$.

Check web using Section B2.4 - "C-Section Webs with Holes under Stress Gradient".

$d_h = 1.5$ in.

$L_h = 4.5$ in.

$h = 4.00 - 2(0.1070 + 0.0713) = 3.643$ in.

Check limits

$d_h/h = 1.5/3.643 = 0.412 < 0.7$ OK

$h/t = 3.643/0.0713 = 51.1 < 200$ OK

Holes are centered at mid-depth of web OK

Clear distance between holes $= 24.0 - 4.5 = 19.5$ in. > 18.0 in. OK

Corner radii = $0.25 \text{ in.} > (2)(0.0713) = 0.143$ in. OK

$d_h < 2.5$ in. OK

$L_h = 4.5$ in. OK

d_h > 9/16 in. OK

Since $d_h/h > 0.38$, treat compression portion of web as a uniformly compressed unstiffened element as follows:

$$w = (h - d_h)/2 = (3.643 - 1.50)/2 = 1.072 \text{ in.}$$

$$k = 0.43$$

Calculate first estimate of f_1 at the top of the flat width using similar triangles with gross properties.

$$f = f_1 = 50\left(\frac{4.00/2 - 0.0713 - 0.1070}{4.00/2}\right) = 45.5 \text{ ksi}$$

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 0.43\frac{\pi^2(29500)}{12(1-0.3^2)}\left(\frac{0.0713}{1.072}\right)^2 = 50.7 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{45.5}{50.7}} = 0.947 > 0.673 \ \therefore \text{ web is subject to local buckling}$$

$$\rho = (1 - 0.22/\lambda)/\lambda \qquad (Eq.\ B2.1\text{-}3)$$

$$= (1 - 0.22/0.947)/0.947 = 0.811$$

$$b = \rho w \qquad (Eq.\ B2.1\text{-}2)$$

$$= (0.811)(1.072) = 0.869 \text{ in.}$$

Since the web is not fully effective, the cross section is not eligible for design using the cold work of forming provision in this area.

Check Flange and Lip

It can be shown that the flange and lip are fully effective at this stress level (calculations not shown).

Recompute Section Properties

Calculate the effective section modulus, S_e, deducting both the 1.50 inch hole and the ineffective portion of the compression area of the web. Using the methods illustrated in the examples in Part I, the effective flexural properties can be computed as:

y_c = 2.03 in. (from top fiber)

I_{xe} = 1.32 in.^{4}

S_{xe} = 0.648 in.^{3}

Further Iterations

The shift in the centroid causes a very slight change to the stress distribution and consequently a very small change in the value of f_1 at the top of the flat width of the web, but not enough to change the values calculated above.

Nominal Flexural Strength

$M_n = S_eF_y$ (*Eq.* C3.1.1-1)

$= (0.648)(50) = 32.4$ kip-in.

Alternatively, M_n can be taken from Table II-2. For a 400S162-68 with F_y = 50 ksi,

M_n = 32.4 kip-in.

d) Available Strength

ASD Allowable Strength

$\Omega_b = 1.67$

At center, away from holes

$$\frac{M_n}{\Omega_b} = \frac{37.9}{1.67} = 22.7 \text{ kip-in.} > 21.6 \text{ kip-in. OK}$$

At holes nearest center

$$\frac{M_n}{\Omega_b} = \frac{32.4}{1.67} = 19.4 \text{ kip-in.} > 17.2 \text{ kip-in. OK}$$

LRFD Design Strength

$\phi_b = 0.95$

At center, away from holes

$\phi_bM_n = (0.95)(37.9) = 36.0$ kip-in. > 33.1 kip-in. OK

At holes nearest center

$\phi_bM_n = (0.95)(32.4) = 30.8$ kip-in. > 26.4 kip-in. OK

2. Shear Strength

a) Required Strength

ASD Required Strength

V = 0.450 kips (from above)

LRFD Required Strength

V_u = 0.690 kips (from above)

b) Shear Strength without Holes - Section C3.2.1

h/t = 51.1 (computed above)

$$\sqrt{Ek_v/F_y} = \sqrt{(29500)(5.34)/50} = 56.1$$

Since $h/t < \sqrt{Ek_v/F_y}$,

$F_v = 0.60F_y$ (*Eq.* C3.2.1-2)

$= (0.60)(50) = 30$ ksi

$V_n = A_wF_v$ (*Eq.* C3.2.1-1)

$= (3.643)(0.0713)(30) = 7.79$ kips

c) Shear Strength with Holes - Section C3.2.2

Limits same as those checked above OK

$$c = h/2 - d_h/2 \qquad (Eq.\ C3.2.2\text{-}3)$$

$$= 3.643/2 - 1.50/2 = 1.07 \text{ in.}$$

$$c/t = 1.07/0.0713 = 15.0$$

Since $5 < c/t < 54$,

$$q_s = c/(54t) \qquad (Eq.\ C3.2.2\text{-}1)$$

$$= 1.07/\left[(54)(0.0713)\right] = 0.278$$

$$V_n = q_s V_n = (0.278)(7.79) = 2.17 \text{ kips}$$

Alternatively, V_n can be taken from Table II-2. For a 400S162-68 with F_y = 50 ksi,

V_n = 2.17 kips

d) Available Strength

ASD Allowable Strength

$\Omega_v = 1.60$

$$\frac{V_n}{\Omega_v} = \frac{2.17}{1.60} = 1.36 \text{ kips} > 0.450 \text{ kips. OK}$$

LRFD Design Strength

$\phi_v = 0.95$

$$\phi_v V_n = (0.95)(2.17) = 2.06 \text{ kips} > 0.690 \text{ kips. OK}$$

3. Combined Bending and Shear Strength

ASD

Near the center of the beam (no holes)

$$\sqrt{\left(\frac{\Omega_b M}{M_{nxo}}\right)^2 + \left(\frac{\Omega_v V}{V_n}\right)^2} \le 1.0 \qquad (Eq.\ C3.3.1\text{-}1)$$

$$\sqrt{\left(\frac{(1.67)(21.6)}{37.9}\right)^2 + \left(\frac{(1.60)(0.450)}{7.79}\right)^2} = 0.956 < 1.0 \text{ OK}$$

At edge of the hole closest to the center

$$\sqrt{\left(\frac{(1.67)(17.2)}{32.4}\right)^2 + \left(\frac{(1.60)(0.450)}{2.17}\right)^2} = 0.947 < 1.0 \text{ OK} \qquad (Eq.\ C3.3.1\text{-}1)$$

Alternatively, this case can be checked with Table II-11a. For a 400S162-68 with F_y = 50 ksi, using a required allowable moment, M, of 17.2 kip-in., conservatively interpolate the maximum permitted shear, V.

for M = 16.8 kip-in., $V \le 0.678$ kips

for M = 18.7 kip-in., $V \le 0.351$ kips

for M = 17.2 kip-in., interpolating,

$$V \le 0.351 + \left(\frac{18.7 - 17.2}{18.7 - 16.8}\right)(0.678 - 0.351) = 0.609 \text{ kips} > 0.450 \text{ kips OK}$$

LRFD

Near the center of the beam (no holes)

$$\sqrt{\left(\frac{\overline{M}}{\phi_b M_{nxo}}\right)^2+\left(\frac{\overline{V}}{\phi_v V_n}\right)^2} \le 1.0 \qquad (Eq.\ C3.3.2\text{-}1)$$

$\overline{M}$ = M_u

$\overline{V}$ = V_u

$$\sqrt{\left(\frac{33.1}{(0.95)(37.9)}\right)^2+\left(\frac{0.690}{(0.95)(7.79)}\right)^2} = 0.924 < 1.0 \text{ OK} \qquad (Eq.\ C3.3.2\text{-}1)$$

At edge of hole closest to the center

$$\sqrt{\left(\frac{26.4}{(0.95)(32.4)}\right)^2+\left(\frac{0.690}{(0.95)(2.17)}\right)^2} = 0.921 < 1.0 \text{ OK} \qquad (Eq.\ C3.3.2\text{-}1)$$

Alternatively, this case can be checked with Table II-11b. For a 400S162-68 with F_y = 50 ksi, using a required moment, M_u, of 26.4 kip-in., conservatively interpolate the maximum permitted factored shear, V_u.

for M_u = 21.8 kip-in., $V_u \le 1.46$ kips

for M_u = 26.7 kip-in., $V_u \le 1.03$ kips

for M_u = 26.4 kip-in., interpolating,

$$V_u \le 1.03+\left(\frac{26.7-26.4}{26.7-21.8}\right)(1.46-1.03) = 1.06 \text{ kips} > 0.690 \text{ kips OK}$$

4. Web Crippling Strength

a) Required Strength

ASD Required Strength

End Condition

P = V = 0.450 kips

Interior Condition

P = $P_D + P_L$ = 0.150 + 0.750 = 0.900 kips

LRFD Required Strength

End Condition

P_u = V_u = 0.690 kips

Interior Condition

$P_u = 1.2P_D + 1.6P_L = (1.2)(0.150)+(1.6)(0.750) = 1.38$ kips

b) Web Crippling Strength without Holes - Section C3.4.1

θ = 90 degrees

R = 0.1070 in.

t = 0.0713 in.

$h = 3.643$ in.

End Condition

$N = 3.0$ in.

From Table C3.4.1-2

Check limits

$h/t = 51.1 < 200$ OK (computed above)

$N/t = 3.0/0.0713 = 42.1 < 210$ OK

$N/h = 3.0/3.643 = 0.823 < 2.0$ OK

For conditions of Fastened to Support/Stiffened or Partially Stiffened Flanges/One Flange Loading/End Condition:

$C = 4$

$C_R = 0.14$

$C_N = 0.35$

$C_h = 0.02$

$\Omega_w = 1.75$

$\phi_w = 0.85$

$R/t = 0.1070/0.0713 = 1.50 < 9$ OK

$$P_n = Ct^2F_y\sin\theta\left(1-C_R\sqrt{\frac{R}{t}}\right)\left(1+C_N\sqrt{\frac{N}{t}}\right)\left(1-C_h\sqrt{\frac{h}{t}}\right) \qquad (Eq.\ C3.4.1\text{-}1)$$

$$= (4)(0.0713)^2(50)\sin(90)\left(1-0.14\sqrt{\frac{0.1070}{0.0713}}\right)\left(1+0.35\sqrt{\frac{3.0}{0.0713}}\right)\left(1-0.02\sqrt{\frac{3.643}{0.0713}}\right)$$

$= 2.36$ kips

Alternatively, P_n can be conservatively interpolated from Table II-14. For a 400S162-68 with $F_y = 50$ ksi, fastened to support, case A:

for N = 2 in., $P_n = 2.06$ kips

for N = 4 in., $P_n = 2.61$ kips

for N = 3 in., interpolating, $P_n = 0.5(2.06 + 2.61) = 2.34$ kips

Interior Condition

$N = 5.0$ in.

From Table C3.4.1-2

Check limits (other limits checked above)

$N/t = 5.0/0.0713 = 70.1 < 210$ OK

$N/h = 5.0/3.643 = 1.37 < 2.0$ OK

For conditions of Fastened to Support/Stiffened or Partially Stiffened Flanges/One Flange Loading/Interior Condition

$C = 13$

$C_R = 0.23$

C_N = 0.14

C_h = 0.01

Ω_w = 1.65

ϕ_w = 0.90

$R/t = 1.50 < 5.0$ OK

$$P_n = Ct^2F_y\sin\theta\left(1-C_R\sqrt{\frac{R}{t}}\right)\left(1+C_N\sqrt{\frac{N}{t}}\right)\left(1-C_h\sqrt{\frac{h}{t}}\right) \quad (Eq.\ C3.4.1\text{-}1)$$

$$= (13)(0.0713)^2(50)\sin(90)\left(1-0.23\sqrt{\frac{0.1070}{0.0713}}\right)\left(1+0.14\sqrt{\frac{5.0}{0.0713}}\right)\left(1-0.01\sqrt{\frac{3.643}{0.0713}}\right)$$

$= 4.79$ kips

Alternatively, P_n can be conservatively interpolated from Table II-14. For a 400S162-68 with F_y = 50 ksi, fastened to support, case B:

for N = 4 in., P_n = 4.51 kips

for N = 6 in., P_n = 5.03 kips

for N = 5 in., interpolating, $P_n = 0.5(4.51+5.03) = 4.77$ kips

c) Web Crippling Strength with Holes - Section C3.4.2

Limits same as those checked above OK

End Condition

x $= 12.0 - 4.50/2 - 3.0/2 = 8.25$ in. (distance between web hole and edge of bearing)

$R_c = 1.01 - 0.325d_h/h + 0.083x/h \le 1.0$ (*Eq.* C3.4.2-1)

$= 1.01 - (0.325)(1.50)/3.643 + (0.083)(8.25)/3.643 = 1.06 > 1$ Use 1.0

$P_n = R_cP_n = (1.0)(2.36) = 2.36$ kips

Alternatively, R_c can be extrapolated from Table II-16b. For stud depth = 4 in., x >> 5 in.,

R_c = 1.00

Interior Condition

x $= 12.0 - 4.50/2 - 5.0/2 = 7.25$ in. (distance between web hole and edge of bearing)

$R_c = 0.90 - 0.047d_h/h + 0.053x/h \le 1.0$ (*Eq.* C3.4.2-2)

$= 0.90 - (0.047)(1.50)/3.643 + (0.053)(7.25)/3.643 = 0.986 < 1.0$ OK

$P_n = R_cP_n = (0.986)(4.79) = 4.72$ kips

Alternatively, R_c can be conservatively interpolated from Table II-16a. For depth = 4 in.,

for x = 4 in., R_c = 0.94

for x = 8 in., R_c = 0.99

for x = 7.25 in., interpolating, $R_c = 0.94 + \left(\frac{7.25-4}{8-4}\right)(0.99-0.94) = 0.98$

d) Available Strength

ASD Allowable Strength

End Condition

$\Omega_w = 1.75$

$$\frac{P_n}{\Omega_w} = \frac{2.36}{1.75} = 1.35 \text{ kips} > 0.450 \text{ kips OK}$$

Interior Condition

$\Omega_w = 1.65$

$$\frac{P_n}{\Omega_w} = \frac{4.72}{1.65} = 2.86 \text{ kips} > 0.900 \text{ kips OK}$$

LRFD Design Strength

End Condition

$\phi_w = 0.85$

$$\phi_w P_n = (0.85)(2.36) = 2.01 \text{ kips} > 0.690 \text{ kips OK}$$

Interior Condition

$\phi_w = 0.90$

$$\phi_w P_n = (0.90)(4.72) = 4.25 \text{ kips} > 1.38 \text{ kips OK}$$

5. Combined Bending and Web Crippling

Concentrated load at center of beam controls

ASD

$$0.91\left(\frac{P}{P_n}\right) + \left(\frac{M}{M_{nxo}}\right) \le \frac{1.33}{\Omega} \qquad (\textit{Eq. } C3.5.1\text{-}1)$$

$$0.91\left(\frac{0.900}{4.72}\right) + \left(\frac{21.6}{37.9}\right) \le \frac{1.33}{1.70}$$

$0.743 < 0.782$ OK

LRFD

$$0.91\left(\frac{\overline{P}}{P_n}\right) + \left(\frac{\overline{M}}{M_{nxo}}\right) \le 1.33\phi \qquad (\textit{Eq. } C3.5.2\text{-}1)$$

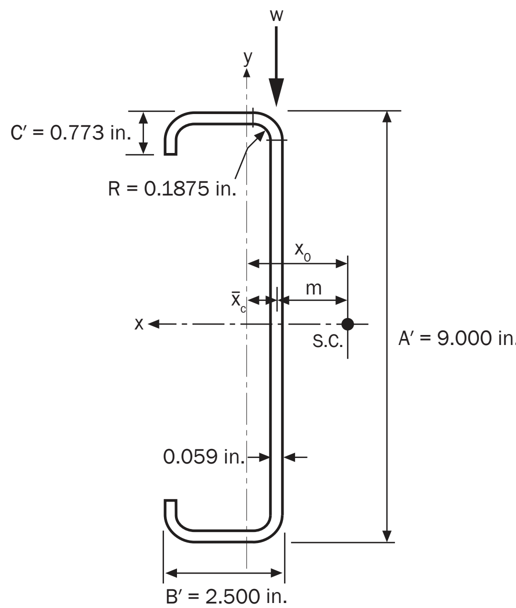
$\overline{P} = P_u$

$\overline{M} = M_u$

$$0.91\left(\frac{1.38}{4.72}\right) + \left(\frac{33.1}{37.9}\right) \le 1.33(0.90)$$

$1.14 < 1.20$ OK

Example II-10: C-Section with Combined Bending and Torsional Loading



Given:

1. Steel: F_y = 55 ksi
2. Section: 9CS2.5x059
3. Gross Section Properties (from Example I-1 or Table I-1)

   $I_x = 10.3\text{ in.}^4$ $S_x = 2.29\text{ in.}^3$ $J = 0.00102\text{ in.}^4$ $C_w = 11.9\text{ in.}^6$

   $x_0 = -1.66\text{ in.}$ $m = 1.05\text{ in.}$ $\bar{x} = 0.641\text{ in.}$

4. Effective Section Properties (from Example I-8 or Table II-1)

   $I_{xe} = 9.18\text{ in.}^4$ $S_{xe} = 1.89\text{ in.}^3$ $\bar{y} = 4.859\text{ in.}$

5. The member is a simply supported beam spanning 25 feet supporting a uniformly distributed load.
6. The load is applied vertically in the plane of the web.
7. The beam has torsional braces at both ends of the member and at the brace points specified below.

Required:

Determine the nominal flexural strength, M_n, based on initiation of yielding of the effective section considering the effects of torsion. Consider alternate conditions of:

1. A single brace at mid-span
2. Two braces, each at the one-third points of the span

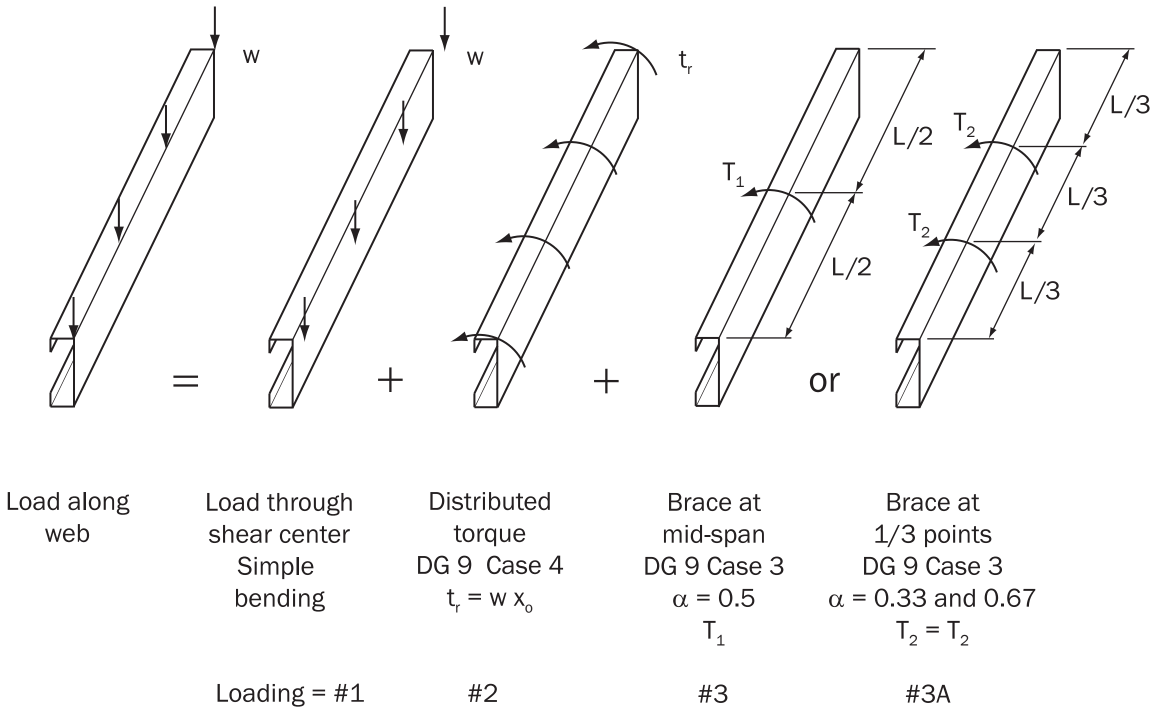
Assumptions:

1. Rotation is completely restrained at the member ends and at the braces.
2. The member is free to warp at both ends.

Solution:

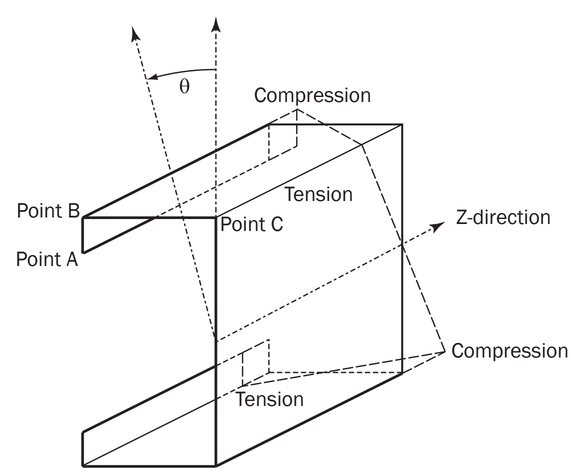
A torsional reduction factor, R, is calculated using Section C3.6 and applied to the nominal strength calculated using Section C3.1.1(a). Note that this reduction factor is not applied to other limit states, such as lateral-torsional buckling or distortional buckling.

This solution is based on the method described in the AISC Steel Design Guide Series 9: "Torsional Analysis of Structural Steel Members"[7] (DG 9). The actual loading is modeled by superimposing the three conditions as shown in the figure below.



Torsional warping stresses are calculated using the second derivative of the angle of rotation, θ, with respect to the position, z, along the length of the member.

The sign convention for use with all torsion expressions are shown in the figure to the right. Note that calculated values for θ and θ'' may be either positive or negative. The proper sign for these calculated values must be used for torsional stress calculations. Calculated positive values are in the directions shown.



Positive rotation and warping stresses

[7] Seaburg, P.A and Carter, C.J, "Torsional Analysis of Structural Steel Members – Steel Design Guide Series 9", American Institute of Steel Construction, Chicago, IL, 1997

For the singly-symmetric channel, only the compression side need be checked for combined bending and warping.

Normal Stresses Due to Warping

$$f_{ws} = EW_{ns}\theta'' \qquad \text{(AISC Design Guide 9, } \textit{Eq}\text{. 4.3a)}$$

where W_{ns} are normalized warping functions (section properties) of the cross-section at each point of consideration given by:

Point A, at tip of flange stiffener

$$W_A = \frac{\bar{a}\left(m - \bar{b}\right)}{2} - \bar{c}\left(m + \bar{b}\right)$$

Point B, at junction of the flange and stiffener

$$W_B = \frac{\bar{a}\left(m - \bar{b}\right)}{2}$$

Point C, at junction of the flange and web

$$W_C = \frac{\bar{a}m}{2}$$

where,

$\bar{a}$ = centerline web height = 8.941 in.

$\bar{b}$ = centerline flange width = 2.441 in.

$\bar{c}$ = centerline lip length = 0.744 in.

m = distance from shear center to web centerline = 1.05 in.

The torsional warping properties for this section are:

$$W_A = \frac{(8.941)(1.05 - 2.441)}{2} - (0.744)(1.05 + 2.441) = -8.82 \text{ in.}^2$$

$$W_B = \frac{(8.941)(1.05 - 2.441)}{2} = -6.22 \text{ in.}^2$$

$$W_C = \frac{(8.941)(1.05)}{2} = 4.69 \text{ in.}^2$$

Formulas for rotation due to a number of torsional loadings are given in Appendix C.4 of DG 9. Summarized below are those used in subsequent calculations.

For Loading #2 above, use DG 9 Case 4 – Uniformly distributed torque on member with pinned ends.

$$\theta_t = \frac{t_r a^2}{GJ}\left[\frac{L^2}{2a^2}\left(\frac{z}{L} - \frac{z^2}{L^2}\right) + \cosh\left(\frac{z}{a}\right) - \tanh\left(\frac{L}{2a}\right)\sinh\left(\frac{z}{a}\right) - 1.0\right]$$

where

$$a = \sqrt{\frac{EC_w}{GJ}}$$

Differentiating twice with respect to z yields

$$\theta_t'' = \frac{t_r}{GJ}\left[-1.0 + \cosh\left(\frac{z}{a}\right) - \tanh\left(\frac{L}{2a}\right)\sinh\left(\frac{z}{a}\right)\right]$$

For Loadings #3 and #3A above, use DG 9 Case 3 – Concentrated torque at αL.

for $0 \le z \le \alpha L$

$$\theta_T = \frac{TL}{GJ}\left[(1.0-\alpha)\frac{z}{L} + \frac{a}{L}\left(\frac{\sinh\left(\frac{\alpha L}{a}\right)}{\tanh\left(\frac{L}{a}\right)} - \cosh\left(\frac{\alpha L}{a}\right)\right)\sinh\left(\frac{z}{a}\right)\right]$$

and

$$\theta_T'' = \frac{T}{aGJ}\left[\left(\frac{\sinh\left(\frac{\alpha L}{a}\right)}{\tanh\left(\frac{L}{a}\right)} - \cosh\left(\frac{\alpha L}{a}\right)\right)\sinh\left(\frac{z}{a}\right)\right]$$

Note that the reduction factor, R, defined in *Eq*. C3.6-1, is a ratio of calculated stresses. These calculated stresses are directly proportional to the value of the applied uniform load. Thus a load of any magnitude can be used to calculate R. In this example, a load of $w = 10$ pounds/foot is used.

1. Mid-Span Bracing

For mid-span bracing, the stresses are maximum at mid-span. Combine Loadings #1, #2 and #3.

Loading #1 - Simple bending through the shear center

$$f_b = \frac{My}{I}$$

$$M = \frac{wL^2}{8} = \frac{10(25)^2(12)}{8(1000)} = 9.38 \text{ kip-in.}$$

Stresses at top flange points A, B and C are all compression stresses.

$$f_{bA} = -\frac{9.38(4.859-0.773)}{9.18} = -4.18\,\text{ksi}$$

$$f_{bB} = f_{bC} = -\frac{9.38(4.859)}{9.18} = -4.97\,\text{ksi}$$

Loading #2 - Uniformly distributed torque – use DG 9 Case 4.

$$\theta_t = \frac{t_r a^2}{GJ}\left[\frac{L^2}{2a^2}\left(\frac{z}{L} - \frac{z^2}{L^2}\right) + \cosh\left(\frac{z}{a}\right) - \tanh\left(\frac{L}{2a}\right)\sinh\left(\frac{z}{a}\right) - 1.0\right]$$

where

$$t_r = \frac{10(1.05)}{12(1000)} = 0.000875 \text{ kip-in./in.}$$

$$a = \sqrt{\frac{29500(11.9)}{11300(0.00102)}} = 175 \text{ in.}$$

$L = 25(12) = 300$ in. $\quad L/a = 300/175 = 1.71$

$z = 150$ in. $\quad z/L = 150/300 = 0.500 \quad z/a = 150/175 = 0.857$

$$\theta_t = \frac{0.000875(175)^2}{11300(0.00102)}\left[\begin{array}{l}\frac{(1.71)^2}{2}\left(0.500-(0.500)^2\right)+\cosh(0.857)\\ -\tanh\left(\frac{1.71}{2}\right)\sinh(0.857)-1.0\end{array}\right]$$

= 0.199 radians

$$\theta_t'' = \frac{0.000875}{11300(0.00102)}\left[-1.0+\cosh(0.857)-\tanh\left(\frac{1.71}{2}\right)\sinh(0.857)\right]$$

= 21.2×10^{-6}

Loading #3 - Brace at Mid-Span - use DG 9 Case 3 with $\alpha = 0.5$.

for $0 \le z \le \alpha L$

$$\theta_T = \frac{TL}{GJ}\left[(1.0-\alpha)\frac{z}{L}+\frac{a}{L}\left(\frac{\sinh\frac{\alpha L}{a}}{\tanh\frac{L}{a}}-\cosh\frac{\alpha L}{a}\right)\sinh\frac{z}{a}\right]$$

and

$$\theta_T'' = \frac{T}{aGJ}\left[\left(\frac{\sinh\frac{\alpha L}{a}}{\tanh\frac{L}{a}}-\cosh\frac{\alpha L}{a}\right)\sinh\frac{z}{a}\right]$$

Set T = 1.0 to find the rotation per kip-in.

$$\theta_T = \frac{1.0(300)}{11300(0.00102)}\left[(1-0.5)0.5+\frac{1}{1.71}\left(\frac{\sinh(0.5(1.71))}{\tanh(1.71)}-\cosh(0.5(1.71))\right)\sinh(0.857)\right]$$

= 1.21 radians

$$\theta_T'' = \frac{1.0}{175(11300)(0.00102)}\left[\left(\frac{\sinh(0.5(1.71))}{\tanh(1.71)}-\cosh(0.5(1.71))\right)\sinh(0.857)\right]$$

= -172×10^{-6}

Calculate the required value of torque provided by mid-span brace to prevent rotation at mid-span.

$$\theta = \theta_t + T_1\theta_T = 0.199 + T_1(1.21) = 0$$

T_1 = -0.164 kip-in.

Using this brace force, combine the calculated values for θ'' from each loading to obtain θ'' for the mid-span braced condition.

$$\theta'' = \theta_t'' + T_1\theta_T'' = -21.2\times10^{-6} - 0.164\left(-172\times10^{-6}\right) = 7.01\times10^{-6}$$

The torsional warping stresses are:

$$f_w = EW_n\theta'' = 29500W_n\left(7.01\times10^{-6}\right) = 0.207W_n$$

$$f_{wA} = 0.207(-8.82) = -1.83 \text{ ksi}$$

$$f_{wB} = 0.207(-6.22) = -1.29 \text{ ksi}$$

$$f_{wC} = 0.207(4.69) = 0.971 \text{ ksi}$$

Determine the location of the maximum combined flexural and warping stress.

$$f_A = f_{bA} + f_{wA} = -4.18 - 1.83 = -6.01 \text{ ksi}$$

$$f_B = f_{bB} + f_{wB} = -4.97 - 1.29 = -6.26 \text{ ksi} \quad \text{CONTROLS}$$

$$f_C = f_{bC} + f_{wC} = -4.97 + 0.971 = -4.00 \text{ ksi}$$

Calculate the reduction factor.

$$R = \frac{f_{bending}}{f_{bending} + f_{torsion}} \qquad (Eq.\ C3.6\text{-}1)$$

$$= \frac{-4.97}{-4.97 - 1.29} = 0.794$$

Note that this value occurs at the intersection of the flange and stiffener; therefore, no increase is permitted.

Calculate the nominal yielding strength.

$$M_n = RS_eF_y$$

$$= (0.794)(1.89)(55) = 82.5 \text{ kip-in.}$$

Other applicable limit states should also be evaluated (not shown).

2. Third-Point Bracing

For this condition, stresses are calculated at both the third-points and at mid-span, since it is not obvious by inspection which location will govern. Superimpose the stresses from Loadings #1, #2 and #3A. Use DG 9 Case 3 to calculate θ and θ″ at these points due to the torsional restraint provided by the braces. The value of the torque at the brace points is calculated by requiring that the value of θ be zero at these two points. Note by symmetry, the torques at the braces are equal.

Loading #1 - Simple bending through the shear center

Flexural stresses mid-span are the same as previously calculated. Those at the third-points are:

$$M = \frac{wL^2}{9} = \frac{10(25)^2 12}{9(1000)} = 8.33 \text{ kip-in.}$$

$$f_{bA} = -\frac{8.33}{9.18}(4.859 - 0.773) = -3.71 \text{ ksi}$$

$$f_{bB} = f_{bC} = -\frac{8.33}{9.18}(4.859) = -4.41 \text{ ksi}$$

Loading #2 - Uniformly Distributed Torque – Use DG 9 Case 4

Values at mid-span are as previously calculated. Those at third-points are:

$$z = L/3 = 100 \text{ in.} \qquad z/L = 0.333 \qquad z/a = 0.571$$

$$\theta_{t1/3} = \frac{0.000875(175)^2}{11300(0.00102)}\left[\begin{array}{l}\frac{(1.71)^2}{2}\left(0.333-(0.333)^2\right)+\cosh(0.571)\\ -\tanh\left(\frac{1.71}{2}\right)\times\sinh(0.571)-1.0\end{array}\right]$$

= 0.173 rads

By symmetry, rotation at the 2/3 point is equal to the rotation at the 1/3 point: $\theta_{t1/3} = \theta_{t2/3}$.

$$\theta''_{t1/3} = \frac{0.000875}{11300(0.00102)}\left[-1.0+\cosh(0.571)-\tanh\left(\frac{1.71}{2}\right)\sinh(0.571)\right]$$

$= -19.0 \times 10^{-6}$

By symmetry, θ'' at the 2/3 point is equal to θ'' at the 1/3 point: $\theta''_{t2/3} = \theta''_{t1/3}$

Loading #3A - Braces at third-points – Use DG 9 Case 3 with $\alpha = 0.667$

Apply the brace torque at 2/3 point and calculate θ_T and θ''_T at z = L/3, z = L/2 and z = 2L/3.

For z = L/3 = 100 and α = 0.667

$$\theta_{T1/3} = \frac{1.0(300)}{11300(0.00102)}\left[(1-0.667)0.333+\frac{1}{1.71}\left(\begin{array}{c}\frac{\sinh(0.667(1.71))}{\tanh(1.71)}\\ -\cosh(0.667(1.71))\end{array}\right)\sinh(0.571)\right]$$

= 0.826 radians

$$\theta''_{T1/3} = \frac{1.0}{175(11300)(0.00102)}\left[\left(\frac{\sinh(0.667(1.71))}{\tanh(1.71)}-\cosh(0.667(1.71))\right)\sinh(0.571)\right]$$

$= -67.1 \times 10^{-6}$

For z = L/2 = 150

$$\theta_{T1/2} = \frac{1.0(300)}{11300(0.00102)}\left[(1-0.667)0.500+\frac{1}{1.71}\left(\begin{array}{c}\frac{\sinh(0.667(1.71))}{\tanh(1.71)}\\ -\cosh(0.667(1.71))\end{array}\right)\sinh(0.857)\right]$$

= 1.03 radians

$$\theta''_{T1/2} = \frac{1.0}{175(11300)(0.00102)}\left[\left(\frac{\sinh(0.667(1.71))}{\tanh(1.71)}-\cosh(0.667(1.71))\right)\sinh(0.857)\right]$$

$= -108 \times 10^{-6}$

For z = 2L/3 = 200

$$\theta_{T2/3} = \frac{1.0(300)}{11300(0.00102)}\left[(1-0.667)0.667+\frac{1}{1.71}\left(\begin{array}{c}\frac{\sinh(0.667(1.71))}{\tanh(1.71)}\\ -\cosh(0.667(1.71))\end{array}\right)\sinh(1.14)\right]$$

= 0.982 radians

$$\theta''_{T2/3} = \frac{1.0}{175(11300)(0.00102)}\left[\left(\frac{\sinh(0.667(1.71))}{\tanh(1.71)} - \cosh(0.667(1.71))\right)\sinh(1.14)\right]$$

$$= -156 \text{ x } 10^{-6}$$

Calculate the value of the torques at third-points required to prevent rotation at those brace points.

$$\theta_{1/3} = \theta_{t1/3} + T_2\theta_{T1/3} + T_2\theta_{T2/3} = 0.173 + 0.826T_2 + 0.982T_2 = 0$$

$$T_2 = -0.0957 \text{ kip-in.}$$

Calculate torsional warping stresses at the $1/3$ and $2/3$ points.

At z = L/3

$$\theta''_{1/3} = \theta''_{t1/3} + T_2\theta''_{T1/3} + T_2\theta''_{T2/3}$$

$$= -19.0\text{x}10^{-6} + (-0.0957)(-67.1\text{x}10^{-6}) + (-0.0957)(-156\text{x}10^{-6})$$

$$= 2.35\text{x}10^{-6}$$

$$f_w = 29500W_n(2.35\text{x}10^{-6}) = 0.0693W_n$$

$$f_{wA} = 0.0693(-8.82) = -0.611 \text{ ksi}$$

$$f_{wB} = 0.0693(-6.22) = -0.431 \text{ ksi}$$

$$f_{wC} = 0.0693(4.69) = 0.325 \text{ ksi}$$

Determine the location of the maximum combined flexural and warping stress.

$$f_A = f_{bA} + f_{wA} = -3.71 - 0.611 = -4.32 \text{ ksi}$$

$f_B = f_{bB} + f_{wB} = -4.41 - 0.431 = -4.84$ ksi CONTROLS

$$f_C = f_{bC} + f_{wC} = -4.41 + 0.325 = -4.09 \text{ ksi}$$

Calculate the reduction factor at the $1/3$ and $2/3$ points.

$$R = \frac{-4.41}{-4.41 - 0.431} = 0.911 \qquad (Eq.\ C3.6\text{-}1)$$

Calculate the nominal yielding strength at the $1/3$ and $2/3$ points.

$$M_n = RS_eF_y$$

$$= (0.911)(1.89)(55) = 94.7 \text{ kip-in.}$$

Calculate torsional warping stresses at mid-span.

At z = L/2

$$\theta''_{1/2} = \theta''_{t1/2} + 2T_2\theta''_{T1/2} = -21.2\text{x}10^{-6} + 2(-0.0957)(-108\text{x}10^{-6})$$

$$= -0.529 \text{ x } 10^{-6}$$

$$f_w = 29500W_n(-0.529\text{x}10^{-6}) = -0.0156W_n$$

$$f_{wA} = -0.0156(-8.82) = 0.138 \text{ ksi}$$

$$f_{wB} = -0.0156(-6.22) = 0.0970 \text{ ksi}$$

$$f_{wC} = -0.0156(4.69) = -0.0732 \text{ ksi}$$

Determine the location of the maximum combined flexural and warping stress.

$$f_A = f_{bA} + f_{wA} = -4.18 + 0.138 = -4.04 \text{ ksi}$$

$$f_B = f_{bB} + f_{wB} = -4.97 + 0.0970 = -4.87 \text{ ksi}$$

$$f_C = f_{bC} + f_{wC} = -4.97 - 0.0732 = -5.04 \text{ ksi} \quad \text{CONTROLS}$$

Calculate the reduction factor at mid-span.

$$R = (1.15)\frac{-4.97}{-4.97 - 0.0732} = 1.13 > 1.0 \qquad (\textit{Eq. } \text{C3.6-1})$$

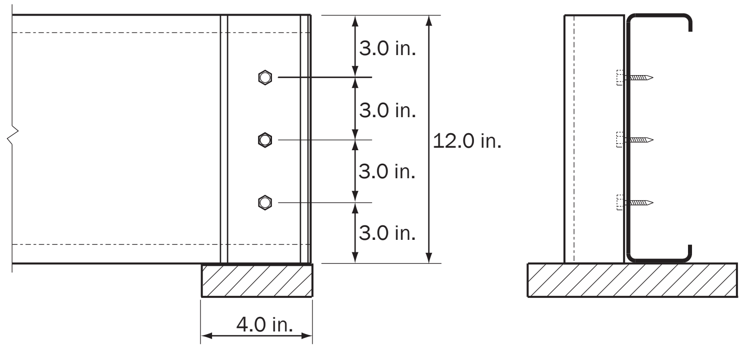
Since R exceeds 1.0, take R as 1.0 at midspan. The 15% increase is permitted since the maximum combined stress occurs at the junction of the flange and web.

Calculate the nominal yielding strength at mid-span.

$$M_n = RS_eF_y$$

$$= (1.0)(1.89)(55) = 104 \text{ kip-in.}$$

Example II-11: Web Crippling



Given:

1. Flexural member: SSMA Stud 1200S200-68 (50 ksi)
2. Bearing stiffener: SSMA Stud 362S162-33 (33 ksi)

Required:

Calculate the available bearing strength of the joist section with the C-section bearing stiffener using both ASD and LRFD

Solution:

Calculate the available ASD and LRFD strength using Section C3.7.

Use Section C3.7.1 if the w/t_s limits for the stiffener are not exceeded.

1. Check Applicability Limits for Section C3.7.1

Check web of stiffener:

$$w/t_s = \frac{D-2(R+t_s)}{t_s}$$

$$= \frac{3.625-2(0.0765+0.0346)}{0.0346} = 98.3$$

$$\text{Limit} = 1.28\sqrt{E/F_y}$$

$$= 1.28\sqrt{29500/33} = 38.3 < 98.3 \quad \text{NG; therefore, try Section C3.7.2}$$

2. Check Applicability Limits for Section C3.7.2

(1) The stiffener has full bearing; therefore, use 100% of the calculated capacity. OK

(2) The stiffener is a C-section with a web depth of 3.625 in. > 3.5 in. minimum. The stiffener has a thickness of 0.0346 in. > 0.0329 in. minimum. OK

(3) The stiffener is attached to the flexural member with three screws. OK

(4) The distance from the flexural member flanges to the first fastener is d/4 > d/8 minimum. OK

(5) The length of the stiffener is equal to the depth of the flexural member. OK

(6) The bearing width is greater than 1 ½ in. OK

2. Calculate nominal strength, P_n, using Section C3.7.2

Calculate the nominal bearing strength.

$$P_n = 0.7\left(P_{wc} + A_e F_y\right) \geq P_{wc} \qquad (\textit{Eq. } \text{C3.7.2-1})$$

From Table II-14 for a 1200S200-68 (50 ksi), Fastened to the support, Case C, N = 4 in.

P_{wc} = 1.26 kips (flexural member)

From Table III-2 for a 362S162-33 (33 ksi)

P_n = 5.72 kips (stiffener) = $A_e F_y$

Nominal Strength

$$P_n = 0.7\left(1.26 + 5.72\right) \geq 1.26 \text{ kips} \qquad (\textit{Eq. } \text{C3.7.2-1})$$

= 4.89 kips > 1.26 kips; therefore, use 4.89 kips

3. Available Strength

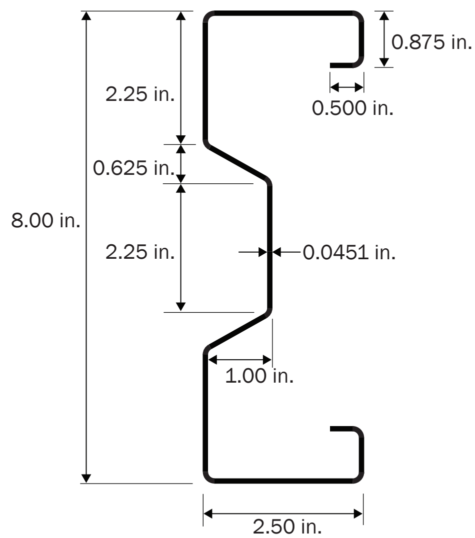
ASD - Allowable strength

$$\frac{P_n}{\Omega} = \frac{4.89}{1.70} = 2.88 \text{ kips} \qquad (\textit{Eq. } \text{A4.1.1-1})$$

LRFD - Design strength

$$\phi P_n = 0.90\left(4.89\right) = 4.40 \text{ kips} \qquad (\textit{Eq. } \text{A5.1.1-1})$$

Example II-12: Web-Stiffened C-Section by the Direct Strength Method – Flexure



Given:

1. Steel: F_y = 50 ksi, F_u = 65 ksi
2. Sigma section (C-section with web stiffener) as shown above
3. The member is a simply supported flexural member fully braced against lateral-torsional buckling.

Required:

Calculate the ASD and LRFD available flexural strengths using the Direct Strength procedure from *Specification* Appendix 1

Solution:

Although the Direct Strength method may be used for any cross-section, it is particularly well suited to this example, since the cross-section is somewhat complex and the *Specification* has no provisions for the complex edge stiffeners on the flanges.

1. Perform a finite strip analysis

A finite strip analysis of the cross-section is performed using a program such as CUFSM[8]. A pure flexural stress distribution is assumed with the extreme fibers at F_y. Results from the analysis include the bending moment under the assumed stress distribution, M_y, and a graph of the section buckling strength versus unbraced length, shown below.

From the analysis:

Yield moment

M_y = 86.4 kip-in.

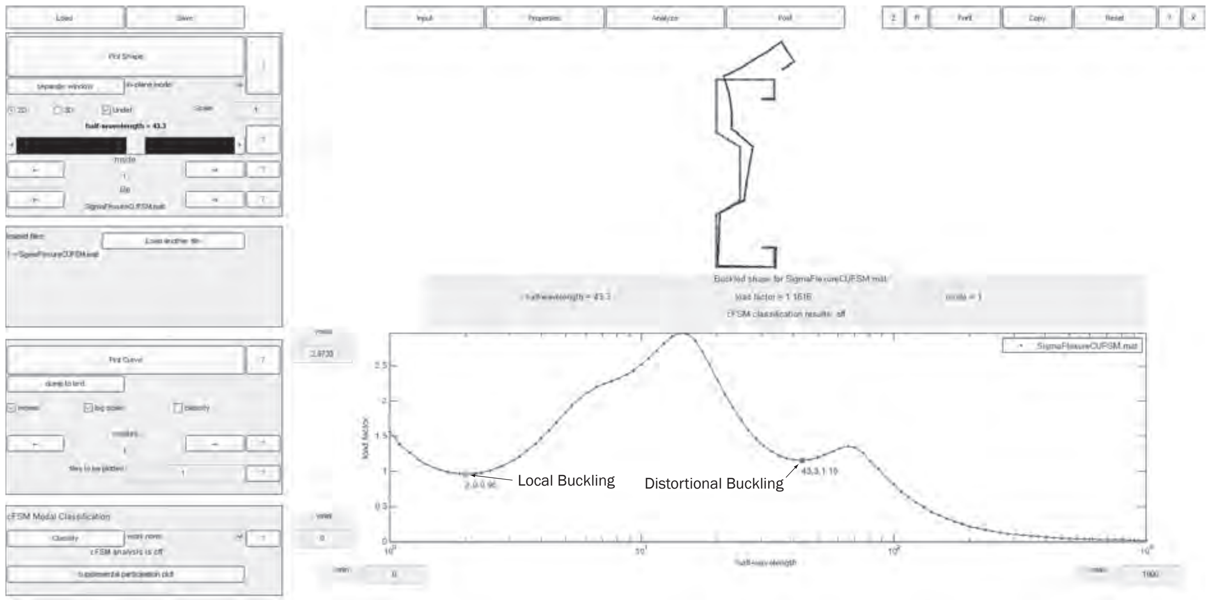
[8] Schafer, B.W., Ádány, S. "Buckling analysis of cold-formed steel members using CUFSM: conventional and constrained finite strip methods." Eighteenth International Specialty Conference on Cold-Formed Steel Structures, Orlando, FL. October 2006. Available at www.ce.jhu.edu/bschafer/cufsm

Critical elastic local buckling moment

$$M_{cr\ell} = 0.96M_y = (0.96)(86.4) = 82.9 \text{ kip-in.}$$

Critical elastic distortional buckling moment

$$M_{crd} = 1.16M_y = (1.16)(86.4) = 100 \text{ kip-in.}$$



2. Calculate the nominal flexural strength

Per section 1.2.2 of Appendix 1, take M_n as the lowest of the nominal strengths for lateral-torsional buckling, M_{ne}, local buckling, $M_{n\ell}$ and distortional buckling, M_{nd}.

1) Lateral-torsional buckling: In this case, since the member is fully braced against lateral-torsional buckling,

$$M_{ne} = M_y = 86.4 \text{ kip-in.} \qquad (Eq.\ 1.2.2\text{-}3)$$

2) Local buckling:

$$\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}} \qquad (Eq.\ 1.2.2\text{-}7)$$

$$= \sqrt{86.4/82.9} = 1.02$$

Since $\lambda_\ell > 0.776$,

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

$$= \left(1 - 0.15\left(\frac{82.9}{86.4}\right)^{0.4}\right)\left(\frac{82.9}{86.4}\right)^{0.4} 86.4 = 72.4 \text{ kip-in.}$$

3) Distortional buckling:

$$\lambda_d = \sqrt{M_y/M_{crd}} \qquad (Eq.\ 1.2.2\text{-}10)$$

$$= \sqrt{86.4/100} = 0.93$$

Since $\lambda_d > 0.673$,

$$M_{n\ell} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

$$= \left(1 - 0.22\left(\frac{100}{86.4}\right)^{0.5}\right)\left(\frac{100}{86.4}\right)^{0.5} 86.4 = 71.0\ \text{kip-in.}$$

4) The nominal flexural strength is therefore 71.0 kip-in, governed by distortional buckling.

3. Calculate the available strengths

Check the limitations for prequalified beams in Table 1.1.1-2 to determine the appropriate strength reduction factors. Since there is no prequalified category for C-sections with web stiffeners and complex lips, use the strength reduction factors from Section A1.2(b)

ASD - Allowable strength

$$\frac{M_n}{\Omega} = \frac{71.0}{2.00} = 35.5\ \text{kip-in.} \qquad (Eq.\ A4.1.1\text{-}1)$$

LRFD - Design strength

$$\phi M_n = 0.80(71.0) = 56.8\ \text{kip-in.} \qquad (Eq.\ A5.1.1\text{-}1)$$

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PART III
COLUMN DESIGN

For Use With the 2007 Edition of the North American Specification for the Design of Cold-Formed Steel Structural Members

PART III - COLUMN DESIGN

The design of cold-formed steel columns requires the consideration of the limit states of:

1. Combined overall member buckling and local buckling, and
2. Distortional buckling

Specification Section C4 includes provisions for the evaluation of these limit states. For columns that are parts of certain structural systems, Section D6 includes provisions that supersede some of the general provisions of Section C4.

Overall and Local Buckling: The strength of all columns is limited by the combined limit state of global and local buckling, which is evaluated using Section C4.1. Although the specifics vary somewhat for different cross-section shapes, the general procedure involves 1) determination of the elastic column buckling stress, 2) transformation of the elastic buckling stress to a critical buckling stress, taking into account the effects of inelasticity and 3) determination of the effective area with the section at the critical buckling stress. See *Manual* Section 3.6 and Examples I-8 and I-10 through I-13 for further information on the calculation of effective area of compression members.

The elastic buckling stress is taken as the lowest of the applicable buckling stresses for flexural (Euler) buckling, torsional buckling and flexural-torsional buckling. All cross-sections are subject to flexural buckling about their principal axes, per Section C4.1.1.

All doubly-symmetric sections and most singly-symmetric sections, such as C-shapes, are subject to flexural-torsional buckling per Section C4.1.2. Unlipped singly-symmetric angles having fully effective areas, A_e, at a stress of F_y are exempt from the flexural-torsional provisions and designed based on flexural buckling about the principal axis. Point-symmetric sections, such as Z-shapes, are subject torsional and flexural buckling per Section C4.1.3.

Section D6 of the *Specification* provides specialized provisions for the flexural-torsional buckling of compression members that are elements of metal roof and wall systems, including through-fastened purlins and girts and standing seam roofs.

Distortional Buckling: The distortional buckling limit state involves the cross-sectional deformations of two or more elements acting as a group, e.g., the rotation of the flange and lip of a C-shape about the web-to-flange junction. The *Specification* provides three levels of provisions for this limit state in Section C4.2. Section C4.2(a) requires a simple calculation using basic cross-section dimensions and produces a conservative, and sometimes very conservative, result. This approach can sometimes be used to quickly establish that distortional buckling is not a controlling limit state. For those cases where the extra work is justified, Section C4.2(b) can be used, which requires considerably more complex calculations, but produces accurate results. Section C4.2(c) provides a framework for the use of computerized numerical methods to evaluate distortional buckling. This approach requires fewer calculations than Section C4.2(b) and is especially useful for cross-sections that do not meet the limits of applicability of the other two approaches. For all three approaches, the general procedure involves 1) determination of the elastic distortional buckling stress, 2) determination of the corresponding elastic buckling force using the gross area of the cross-section and 3) transformation of the elastic buckling force to a nominal axial strength, taking into account the effects of inelasticity and post-buckling strength.

For members whose required strengths are determined by first-order analysis, combined flexure and axial force must be checked using Section C5. Alternatively, Appendix 2 permits the use of second-order analysis for the determination of required strengths. In this case, Section C5 is still used to evaluate members subject to combined flexure and axial force, but the moment modifiers and effective length factors used in Section C5 are set to unity.

SECTION 1 - CONCENTRICALLY LOADED COLUMNS

1.1 Notes On The Tables

(a) With the exception of the SSMA studs and tracks, the sections listed in these tables are not necessarily stock sections. They are included primarily as a guide in the design of cold-formed steel structural members.

(b) The section designations listed in these tables correspond to those for which dimensions and properties are given in Tables I-1, I-2 and I-3.

(c) Tabulated properties and capacities are shown to three significant figures.

(d) Where they apply, the algebraic formulae presented in Section 3 of Part I formed the basis of the calculations for these tables.

(e) The strengths listed in Tables III-1 to III-9 inclusive were computed using the yield stress listed in the tables. Cold work of forming increases were not included.

(f) Tables III-1, III-2 and III-3 give the nominal axial strength, P_n, for fully braced C-sections at the yield stress listed in the respective tables. Distortional buckling is not considered.

(g) The values labeled P_{web}, P_{flange} and P_{lip} in Tables III-1, III-2 and III-3 are the highest nominal forces at which the web, flange and lip (if applicable) respectively are fully effective. These values are only meaningful where they do not exceed P_{no} for the section and yield stress in question. A value of 0.00 for P_{web} in Table III-2 indicates that a reduction in web area is required at any stress level when standard punchouts are used.

(h) Tables III-4, III-5 and III-6 give tabulated critical buckling lengths, stiffness coefficients, elastic buckling stresses and nominal axial strengths for the limit state of distortional buckling for use with Section C4.2(b). Rotational restraint from sheathing or discrete bracing is not considered in the values given for the stiffness coefficients, elastic buckling stresses and nominal flexural strengths. To incorporate the strength increases resulting from significant continuous rotational bracing or discrete distortional bracing spaced at less than L_{cr}, use the provisions of Section C4.2(b) or C4.2(c).

(i) Tables III-7, III-8 and III-9 give the nominal axial strength, P_n, for C-sections with varying x- and y-axis unbraced lengths. In all cases, the torsional unbraced length is assumed to equal the y-axis unbraced length and $K_y = K_t = 1.0$. Lengths are arbitrarily cut off at a KL/r_x ratio of approximately 100.

(j) The calculated values in Tables III-1 through III-9 are nominal strengths. These values must be modified by a safety factor, Ω_c, for ASD or a resistance factor ϕ_c, for LRFD. See the appropriate *Specification* section for more information.

(k) The effects of standard factory punchouts in SSMA studs have been included in Tables III-2 and III-8. These punchouts are considered in SSMA studs with flange widths of 1.625 in. or less. Standard punchout sizes are 1.5 in. by 4.5 in. for sections with depths of 3.5 inches or more and 0.75 in. by 4.5 in. for sections with shallower depths.

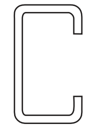
(l) Dashes in the place of data values in the P_n columns of Tables III-2, III-3 and III-5 indicate that the section is not available in the listed grade of steel. Blank data values in Tables III-7, III-8 and III-9 indicate that the section is not available in the listed grade of steel or that KL/r_y exceeds 200.

1.2 Nominal Axial Strength Tables - Braced Columns

Table III - 1

Braced Column Properties [3]
C-Sections With Lips

$\Omega_c = 1.80$
$\phi_c = 0.85$



Section	P_n at $f=F_y$ kips [1], F_y = 33 ksi	P_n at $f=F_y$ kips [1], F_y = 55 ksi	Maximum Effective Force, kips [2] P_{web}	P_{flange}	P_{lip}
12CS4x105	47.6	64.4	8.99	45.8	33.8
12CS4x085	33.1	45.6	4.74	26.0	20.2
12CS4x070	24.3	33.3	2.63	15.6	12.9
12CS3.5x105	46.2	63.5	8.56	54.0	41.2
12CS3.5x085	33.0	44.9	4.51	30.8	24.5
12CS3.5x070	24.0	33.0	2.51	18.5	15.6
12CS2.5x105	40.9	62.0	7.70	82.2	70.1
12CS2.5x085	30.5	44.8	4.06	47.0	41.2
12CS2.5x070	23.4	31.8	2.25	28.3	25.9
10CS4x105	47.1	63.9	12.0	45.3	33.3
10CS4x085	32.9	45.3	6.29	25.8	20.0
10CS4x070	24.2	33.1	3.49	15.4	12.7
10CS4x065	21.5	29.4	2.79	12.7	10.8
10CS3.5x105	45.7	63.0	11.3	53.5	40.7
10CS3.5x085	32.7	44.7	5.96	30.5	24.2
10CS3.5x070	23.8	32.9	3.31	18.4	15.4
10CS3.5x065	21.3	29.3	2.64	15.2	13.0
10CS2.5x105	40.4	61.5	10.1	81.7	69.6
10CS2.5x085	30.2	44.6	5.29	46.7	40.9
10CS2.5x070	23.3	31.7	2.94	28.2	25.8
10CS2.5x065	21.1	28.0	2.35	23.3	21.7
10CS2x105	36.9	55.7	9.43	109	104
10CS2x085	27.4	41.6	4.96	62.5	60.3
10CS2x070	20.9	32.0	2.75	38.0	37.7
10CS2x065	18.9	29.0	2.20	31.1	31.6
9CS2.5x105	40.1	61.1	11.8	81.4	69.3
9CS2.5x085	30.0	44.4	6.20	46.5	40.8
9CS2.5x070	23.2	31.6	3.44	28.1	25.7
9CS2.5x065	21.0	28.0	2.75	23.2	21.7
9CS2.5x059	18.2	24.3	2.05	18.1	17.4
8CS4x105	46.3	63.1	17.2	44.6	32.5
8CS4x085	32.5	44.9	9.05	25.4	19.5
8CS4x070	23.9	32.9	5.01	15.2	12.5
8CS4x065	21.3	29.2	4.00	12.5	10.6
8CS4x059	18.3	25.0	2.98	9.73	8.56
8CS3.5x105	44.9	62.2	16.2	52.7	27.3
8CS3.5x085	32.3	44.3	8.51	30.1	16.7
8CS3.5x070	23.6	32.7	4.72	18.2	11.0
8CS3.5x065	21.1	29.1	3.77	15.0	9.38
8CS3.5x059	18.2	25.0	2.81	11.7	7.68
8CS2.5x105	39.6	60.7	14.2	80.9	64.5
8CS2.5x085	29.8	44.2	7.44	46.3	38.3
8CS2.5x070	23.0	31.4	4.13	28.0	24.3
8CS2.5x065	20.9	27.9	3.29	23.1	20.6
8CS2.5x059	18.1	24.2	2.46	18.1	16.6
8CS2x105	36.2	54.9	13.2	108	103
8CS2x085	27.0	41.2	6.91	62.1	59.9
8CS2x070	20.7	31.8	3.83	37.7	37.4
8CS2x065	18.7	28.8	3.06	30.9	31.4
8CS2x059	16.5	25.1	2.28	23.3	25.0
7CS4x105	45.7	62.5	21.7	44.0	32.0
7CS4x085	32.2	44.6	11.4	25.1	19.2
7CS4x070	23.8	32.7	6.28	15.0	12.3
7CS4x065	21.2	29.1	5.02	12.4	10.5
7CS4x059	18.2	24.9	3.74	9.63	8.46
7CS2.5x105	39.1	60.1	17.6	80.4	68.3
7CS2.5x085	29.5	43.9	9.22	46.0	40.2
7CS2.5x070	22.9	31.3	5.10	27.8	25.4
7CS2.5x065	20.8	27.7	4.07	23.0	21.4
7CS2.5x059	18.0	24.1	3.03	18.0	17.2
6CS4x105	45.0	61.7	28.5	43.2	31.2
6CS4x085	31.8	44.2	14.9	24.7	18.8
6CS4x070	23.5	32.5	8.23	14.8	12.1
6CS4x065	21.0	28.9	6.57	12.2	10.3
6CS4x059	18.1	24.8	4.89	9.50	8.33
6CS2.5x105	38.3	59.4	22.8	79.6	67.5
6CS2.5x085	29.1	43.5	11.9	45.6	39.8
6CS2.5x070	22.6	31.1	6.58	27.6	25.2
6CS2.5x065	20.6	27.6	5.25	22.8	21.2
6CS2.5x059	17.8	24.0	3.91	17.8	17.1
4CS4x105	41.1	58.8	51.6	39.3	27.3
4CS4x085	30.2	42.7	27.8	22.7	16.7
4CS4x070	22.7	31.6	15.8	13.8	11.0
4CS4x065	20.3	28.2	12.8	11.4	9.38
4CS4x059	17.6	24.3	9.70	8.92	7.68
4CS2.5x105	34.4	56.4	47.7	76.7	64.5
4CS2.5x085	27.6	41.9	24.7	44.0	38.3
4CS2.5x070	21.8	30.2	13.6	26.7	24.3
4CS2.5x065	19.9	26.9	10.8	22.1	20.6
4CS2.5x059	17.3	23.5	8.02	17.3	16.6
4CS2x105	31.6	51.7	43.8	103	104
4CS2x085	25.3	39.8	22.7	58.4	59.5
4CS2x070	19.9	31.3	12.5	33.8	35.0
4CS2x065	18.2	28.4	9.92	27.4	28.6
4CS2x059	16.1	24.5	7.37	20.7	21.8

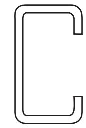
Notes:

1. Axial strengths given are nominal strengths. To obtain the available strength, these values must be modified by safety factors (ASD) or resistance factors (LRFD).
2. P_{web}, P_{flange} and P_{lip} are the highest nominal axial compression forces at which the web, flange and lip, respectively, are fully effective.
3. The distortional buckling limit state is not considered in this table. Distortional buckling strengths are provided in Table III-4.

Table III - 2

Braced Column Properties [3]
SSMA Studs
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85



Section	P_n at $f=F_y$ kips [1]		Maximum Effective Force, kips [2]			Section	P_n at $f=F_y$ kips [1]		Maximum Effective Force, kips [2]		
	F_y						F_y				
	33 ksi	50 ksi	P_{web}	P_{flange}	P_{lip}		33 ksi	50 ksi	P_{web}	P_{flange}	P_{lip}
1200S250-97	37.4	49.9	6.74	60.2	41.3	800S137-97	21.9	30.4	0.00	84.3	56.0
1200S250-68	22.1	28.2	2.29	24.2	19.1	800S137-68	13.5	19.0	0.00	31.7	21.2
1200S250-54*	15.3	19.7	1.14	13.6	12.1	800S137-54	9.93	13.5	0.00	17.2	12.3
1200S200-97	34.0	47.2	6.35	80.4	59.3	800S137-43	7.38	-	0.00	9.64	7.53
1200S200-68	20.8	28.2	2.16	31.5	26.5	800S137-33*	5.04	-	0.00	5.07	4.40
1200S200-54*	15.2	19.2	1.07	17.7	16.6	600S250-97	35.1	47.5	19.4	57.9	38.9
1200S162-97	25.1	35.0	0.00	80.7	56.1	600S250-68	21.4	27.4	6.45	23.4	18.3
1200S162-68	15.5	21.9	0.00	30.8	23.4	600S250-54	14.9	19.3	3.17	13.2	11.8
1200S162-54*	11.4	15.6	0.00	17.0	14.2	600S250-43	11.0	-	1.59	7.61	7.76
1000S250-97	36.9	49.4	8.74	59.8	40.8	600S200-97	31.7	44.8	17.7	78.1	56.9
1000S250-68	22.0	28.0	2.96	24.0	19.0	600S200-68	20.0	27.5	5.90	30.8	25.7
1000S250-54	15.2	19.6	1.47	13.5	12.1	600S200-54	14.8	18.9	2.90	17.3	16.2
1000S250-43*	11.2	-	0.739	7.76	7.92	600S200-43	10.8	-	1.45	9.46	10.4
1000S200-97	33.6	46.7	8.18	79.9	58.8	600S200-33	7.06	-	0.657	4.26	5.64
1000S200-68	20.6	28.1	2.77	31.4	26.3	600S162-97	23.5	33.4	0.00	79.2	54.5
1000S200-54	15.1	19.2	1.38	17.6	16.5	600S162-68	15.0	21.3	0.00	30.3	22.9
1000S200-43*	10.9	-	0.693	9.61	10.6	600S162-54	11.2	15.3	0.00	16.7	14.0
1000S162-97	24.8	34.7	0.00	80.5	55.9	600S162-43	8.45	-	0.00	9.61	8.93
1000S162-68	15.4	21.8	0.00	30.7	23.3	600S162-33	5.84	-	0.00	4.88	5.39
1000S162-54	11.4	15.5	0.00	16.9	14.2	600S137-97	21.0	29.5	0.00	83.4	55.1
1000S162-43*	8.55	-	0.00	9.72	9.04	600S137-68	13.2	18.7	0.00	31.4	20.9
800S250-97	36.3	48.7	12.2	59.1	40.1	600S137-54	9.79	13.4	0.00	17.1	12.2
800S250-68	21.7	27.8	4.11	23.8	18.7	600S137-43	7.30	-	0.00	9.57	7.46
800S250-54	15.1	19.5	2.04	13.4	11.9	600S137-33	5.00	-	0.00	5.04	4.36
800S250-43	11.1	-	1.02	7.71	7.86	550S162-68	14.8	21.2	0.00	30.1	22.8
800S200-97	32.9	46.0	11.3	79.3	58.1	550S162-54	11.1	15.3	0.00	16.7	13.9
800S200-68	20.4	27.9	3.82	31.2	26.1	550S162-43	8.42	-	0.00	9.58	8.91
800S200-54	15.0	19.1	1.89	17.5	16.4	550S162-33	5.83	-	0.00	4.86	5.38
800S200-43	10.9	-	0.949	9.55	10.5	400S200-68	19.1	26.6	11.5	29.9	24.9
800S200-33*	7.10	-	0.429	4.30	5.68	400S200-54	14.4	18.5	5.60	16.9	15.8
800S162-97	24.4	34.2	0.00	80.0	55.4	400S200-43	10.6	-	2.78	9.26	10.2
800S162-68	15.3	21.6	0.00	30.5	23.2	400S200-33	6.97	-	1.26	4.17	5.55
800S162-54	11.3	15.5	0.00	16.9	14.1	400S162-68	14.1	20.5	0.00	29.4	22.1
800S162-43	8.52	-	0.00	9.68	9.00	400S162-54	10.8	14.9	0.00	16.3	13.6
800S162-33*	5.87	-	0.00	4.91	5.42	400S162-43	8.26	-	0.00	9.42	8.74
						400S162-33	5.75	-	0.00	4.79	5.30

Table III - 2

Braced Column Properties [3]
SSMA Studs
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85

Section	P_n at $f=F_y$ kips [1]		Maximum Effective Force, kips [2]			Section	P_n at $f=F_y$ kips [1]		Maximum Effective Force, kips [2]		
	F_y						F_y				
	33 ksi	50 ksi	P_{web}	P_{flange}	P_{lip}		33 ksi	50 ksi	P_{web}	P_{flange}	P_{lip}
400S137-68	12.4	17.9	0.00	30.6	20.1	350S162-68	13.5	20.0	0.00	28.9	21.5
400S137-54	9.40	13.0	0.00	16.7	11.8	350S162-54	10.6	14.7	0.00	16.1	13.3
400S137-43	7.08	-	0.00	9.38	7.27	350S162-43	8.14	-	0.00	9.31	8.63
400S137-33	4.90	-	0.00	4.96	4.28	350S162-33	5.70	-	0.00	4.74	5.25
362S200-68	18.9	26.4	13.7	29.7	24.6	250S162-68	12.9	19.5	0.00	28.5	21.1
362S200-54	14.3	18.3	6.63	16.8	15.6	250S162-54	10.4	14.5	0.00	15.9	13.1
362S200-43	10.5	-	3.29	9.20	10.2	250S162-43	8.06	-	0.00	9.22	8.54
362S200-33	6.94	-	1.48	4.14	5.52	250S162-33	5.66	-	0.00	4.70	5.21
362S162-68	13.8	20.2	0.00	29.1	21.7	250S137-68	11.1	16.8	0.00	29.7	19.2
362S162-54	10.6	14.8	0.00	16.2	13.4	250S137-54	8.97	12.6	0.00	16.3	11.3
362S162-43	8.18	-	0.00	9.34	8.66	250S137-43	6.86	-	0.00	9.17	7.06
362S162-33	5.72	-	0.00	4.76	5.27	250S137-33	4.80	-	0.00	4.87	4.19
362S137-68	12.0	17.5	0.00	30.2	19.8						
362S137-54	9.23	12.8	0.00	16.5	11.6						
362S137-43	7.00	-	0.00	9.29	7.18						
362S137-33	4.87	-	0.00	4.92	4.24						

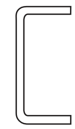
Notes:

1. Axial strengths given are nominal strengths. To obtain the available strength, these values must be modified by safety factors (ASD) or resistance factors (LRFD).
2. P_{web}, P_{flange} and P_{lip} are the highest nominal axial compression forces at which the web, flange and lip, respectively, are fully effective.
3. The distortional buckling limit state is not considered in this table. Distortional buckling strengths are provided in Table III-5.

Table III - 3

Braced Column Properties
SSMA Tracks
C-Sections Without Lips

Ω_c = 1.80
ϕ_c = 0.85

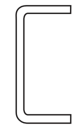


Section	P_n at $f=F_y$ kips [1]		Maximum Effective Force, kips [2]		Section	P_n at $f=F_y$ kips [1]		Maximum Effective Force, kips [2]	
	F_y					F_y			
	33 ksi	50 ksi	P_{web}	P_{flange}		33 ksi	50 ksi	P_{web}	P_{flange}
1200T200-97	29.1	37.7	5.79	19.5	600T150-97	25.7	34.3	13.3	26.7
1200T200-68	15.1	19.3	1.98	6.29	600T150-68	14.0	18.2	4.53	8.35
1200T200-54*	9.77	12.4	0.987	3.05	600T150-54	9.22	11.9	2.25	3.99
1200T150-97	27.8	36.5	5.43	28.8	600T150-43	6.06	-	1.13	1.96
1200T150-68	14.7	18.9	1.86	9.07	600T150-33	3.74	-	0.513	0.881
1200T150-54*	9.59	12.2	0.925	4.35	600T150-30	3.10	-	0.377	0.645
1200T125-97	26.1	35.2	5.24	37.3	600T150-27*	2.59	-	0.281	0.481
1200T125-68	14.4	18.6	1.79	11.5	600T125-97	24.0	33.0	12.6	35.1
1200T125-54*	9.43	12.1	0.894	5.47	600T125-68	13.7	17.9	4.28	10.8
1000T200-97	28.7	37.2	7.33	19.1	600T125-54	9.07	11.7	2.13	5.10
1000T200-68	15.0	19.2	2.50	6.15	600T125-43	5.99	-	1.07	2.49
1000T200-54	9.69	12.3	1.25	2.98	600T125-33	3.71	-	0.485	1.12
1000T200-43*	6.29	-	0.629	1.48	600T125-30	3.07	-	0.356	0.819
1000T150-97	27.4	36.0	6.80	28.4	600T125-27*	2.57	-	0.266	0.610
1000T150-68	14.6	18.8	2.32	8.93	550T200-68	14.3	18.5	5.68	5.38
1000T150-54	9.51	12.2	1.16	4.28	550T200-54	9.34	12.0	2.81	2.59
1000T150-43*	6.21	-	0.584	2.11	550T200-43	6.11	-	1.41	1.28
1000T125-97	25.7	34.8	6.54	36.9	550T200-33	3.77	-	0.638	0.573
1000T125-68	14.3	18.5	2.24	11.4	550T150-68	13.9	18.1	5.11	8.21
1000T125-54	9.36	12.0	1.11	5.39	550T150-54	9.16	11.8	2.54	3.93
1000T125-43*	6.13	-	0.562	2.64	550T150-43	6.03	-	1.28	1.93
800T200-97	28.1	36.6	9.89	18.4	550T150-33	3.73	-	0.578	0.866
800T200-68	14.8	19.0	3.37	5.93	550T150-30	3.09	-	0.424	0.634
800T200-54	9.59	12.2	1.68	2.87	550T150-27	2.58	-	0.317	0.472
800T200-43	6.24	-	0.846	1.42	550T125-68	13.6	17.8	4.81	10.6
800T200-33*	3.82	-	0.383	0.639	550T125-54	9.00	11.6	2.39	5.04
800T150-97	26.8	35.4	9.06	27.8	550T125-43	5.95	-	1.20	2.46
800T150-68	14.4	18.6	3.09	8.71	550T125-33	3.69	-	0.544	1.10
800T150-54	9.40	12.0	1.54	4.17	550T125-30	3.06	-	0.399	0.808
800T150-43	6.15	-	0.775	2.05	550T125-27	2.57	-	0.298	0.602
800T150-33*	3.78	-	0.351	0.922	400T200-68	13.7	17.8	8.40	4.53
800T125-97	25.1	34.1	8.65	36.2	400T200-54	9.04	11.7	4.13	2.18
800T125-68	14.1	18.3	2.95	11.1	400T200-43	5.96	-	2.07	1.07
800T125-54	9.25	11.9	1.47	5.29	400T200-33	3.70	-	0.935	0.482
800T125-43	6.08	-	0.740	2.58	400T150-68	13.3	17.5	8.02	7.53
800T125-33*	3.75	-	0.335	1.16	400T150-54	8.86	11.5	3.95	3.57
600T200-97	27.0	35.5	14.8	17.4	400T150-43	5.87	-	1.98	1.74
600T200-68	14.4	18.6	5.04	5.56	400T150-33	3.66	-	0.895	0.780
600T200-54	9.40	12.0	2.50	2.69	400T150-30	3.04	-	0.657	0.571
600T200-43	6.15	-	1.26	1.33	400T150-27	2.55	-	0.491	0.425
600T200-33	3.78	-	0.570	0.598					

Table III - 3

Braced Column Properties
SSMA Tracks
C-Sections Without Lips

Ω_c = 1.80
ϕ_c = 0.85



Section	P_n at $f=F_y$ kips [1]		Maximum Effective Force, kips [2]		Section	P_n at $f=F_y$ kips [1]		Maximum Effective Force, kips [2]	
	F_y					F_y			
	33 ksi	50 ksi	P_{web}	P_{flange}		33 ksi	50 ksi	P_{web}	P_{flange}
400T125-68	13.0	17.1	7.50	10.03	350T125-68	12.6	16.8	9.11	9.70
400T125-54	8.70	11.3	3.71	4.74	350T125-54	8.54	11.2	4.50	4.58
400T125-43	5.80	-	1.86	2.31	350T125-43	5.72	-	2.26	2.22
400T125-33	3.62	-	0.845	1.035	350T125-33	3.59	-	1.02	1.00
400T125-30	3.01	-	0.620	0.757	350T125-30	2.98	-	0.750	0.727
400T125-27	2.53	-	0.463	0.564	350T125-27	2.51	-	0.560	0.542
400T125-18*	1.21	-	0.136	0.165	350T125-18	1.21	-	0.164	0.158
362T200-68	13.4	17.6	9.46	4.32	250T200-68	11.9	16.5	14.8	3.68
362T200-54	8.92	11.6	4.64	2.08	250T200-54	8.36	11.0	7.21	1.77
362T200-43	5.91	-	2.32	1.02	250T200-43	5.63	-	3.58	0.872
362T200-33	3.67	-	1.05	0.460	250T200-33	3.54	-	1.62	0.392
362T150-68	13.0	17.2	9.08	7.13	250T150-68	11.6	16.1	14.4	5.91
362T150-54	8.74	11.4	4.46	3.38	250T150-54	8.18	10.8	7.03	2.80
362T150-43	5.82	-	2.23	1.65	250T150-43	5.54	-	3.49	1.37
362T150-33	3.63	-	1.01	0.739	250T150-33	3.50	-	1.58	0.613
362T150-30	3.02	-	0.741	0.54	250T150-30	2.92	-	1.16	0.449
362T150-27	2.53	-	0.554	0.403	250T150-27	2.46	-	0.866	0.334
362T125-68	12.7	16.9	8.65	9.79	250T125-68	11.2	15.8	14.1	8.17
362T125-54	8.59	11.2	4.28	4.62	250T125-54	8.03	10.7	6.87	3.82
362T125-43	5.74	-	2.15	2.25	250T125-43	5.46	-	3.42	1.85
362T125-33	3.60	-	0.973	1.01	250T125-33	3.47	-	1.55	0.829
362T125-30	2.99	-	0.714	0.738	250T125-30	2.90	-	1.14	0.606
362T125-27	2.51	-	0.534	0.550	250T125-27	2.44	-	0.848	0.451
362T125-18	1.21	-	0.157	0.160	250T125-18	1.19	-	0.249	0.131
350T200-68	13.3	17.5	9.87	4.25	162T125-33	3.24	-	2.62	0.684
350T200-54	8.88	11.5	4.84	2.04	162T125-30	2.73	-	1.92	0.500
350T200-43	5.88	-	2.42	1.01	162T125-27	2.32	-	1.43	0.372
350T200-33	3.66	-	1.09	0.452	162T125-18	1.15	-	0.422	0.108
350T150-68	13.0	17.1	9.49	6.99					
350T150-54	8.70	11.3	4.66	3.31					
350T150-43	5.79	-	2.33	1.62					
350T150-33	3.62	-	1.05	0.725					
350T150-30	3.01	-	0.774	0.530					
350T150-27	2.53	-	0.578	0.395					

Notes:

1. Axial strengths given are nominal strengths. To obtain the available strength, these values must be modified by safety factors (ASD) or resistance factors (LRFD).
2. P_{web} and P_{flange} are the highest nominal axial compression forces at which the web and flange, respectively, are fully effective.

1.3 Distortional Buckling Axial Strength Tables

Tables III-4, III-5 and III-6 provide computed distortional buckling properties and strengths under axial load for the representative C-shapes, SSMA studs and Z-shapes with lips, respectively. The values in these tables have been calculated for use with Section C4.2(b).

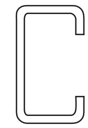
(a) Where a known rotational stiffness, k_ϕ, from bracing or sheathing is available, the values in the columns under the headings $k_{\phi fe}$, $\tilde{k}_{\phi fg}$, $k_{\phi we}$ and $\tilde{k}_{\phi wg}$ may be used in *Eq.* C4.2-10 to calculate a more exact value of F_d.

(b) The values in the column under the heading P_n are valid for cases where bracing against distortional buckling is insignificant or spaced at a length equal to or greater than L_{cr}.

Table III – 4

Distortional Buckling Properties
Axial Strength
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85

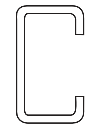


Section	Per Section 4.2(b)							
	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d ksi	P_n[1] $_{(L_m \geq L_{cr})}$ (F_y=33 ksi) kips	P_n[1] $_{(L_m \geq L_{cr})}$ (F_y=55 ksi) kips
12CS4x105	32.7	0.712	0.0322	0.521	0.0280	20.5	44.3	57.6
12CS4x085	35.2	0.363	0.0223	0.277	0.0195	15.3	31.2	40.2
12CS4x070	37.8	0.196	0.0158	0.154	0.0139	11.8	22.6	28.9
12CS3.5x105	30.2	0.721	0.0267	0.521	0.0328	20.9	42.5	55.4
12CS3.5x085	32.5	0.367	0.0184	0.277	0.0229	15.6	30.0	38.6
12CS3.5x070	35.0	0.198	0.0130	0.154	0.0162	12.0	21.7	27.8
12CS2.5x105	24.6	0.750	0.0175	0.521	0.0492	19.1	36.7	47.6
12CS2.5x085	26.6	0.380	0.0119	0.277	0.0342	14.2	25.8	33.1
12CS2.5x070	28.7	0.204	0.00838	0.154	0.0242	11.0	18.7	23.8
10CS4x105	31.2	0.835	0.0353	0.625	0.0177	27.5	45.7	60.3
10CS4x085	33.6	0.427	0.0244	0.332	0.0124	20.6	32.5	42.3
10CS4x070	36.1	0.231	0.0173	0.185	0.00881	16.0	23.7	30.6
10CS4x065	37.2	0.183	0.0151	0.148	0.00772	14.5	21.0	27.0
10CS3.5x105	28.8	0.845	0.0293	0.625	0.0208	29.4	44.4	58.9
10CS3.5x085	31.1	0.431	0.0202	0.332	0.0145	22.0	31.7	41.4
10CS3.5x070	33.4	0.233	0.0143	0.185	0.0103	17.0	23.2	30.0
10CS3.5x065	34.4	0.184	0.0125	0.148	0.00902	15.5	20.6	26.5
10CS2.5x105	23.5	0.877	0.0192	0.625	0.0312	29.8	39.7	52.7
10CS2.5x085	25.4	0.445	0.0131	0.332	0.0217	22.4	28.4	37.1
10CS2.5x070	27.4	0.240	0.00917	0.185	0.0153	17.3	20.8	26.9
10CS2.5x065	28.2	0.189	0.00800	0.148	0.0134	15.8	18.4	23.7
10CS2x105	20.5	0.905	0.0152	0.625	0.0411	27.2	35.8	47.3
10CS2x085	22.2	0.457	0.0103	0.332	0.0284	20.4	25.5	33.2
10CS2x070	24.0	0.245	0.00715	0.185	0.0201	15.8	18.6	24.0
10CS2x065	24.7	0.193	0.00622	0.148	0.0176	14.4	16.5	21.2
9CS2.5x105	22.9	0.960	0.0202	0.695	0.0240	37.5	40.8	54.9
9CS2.5x085	24.7	0.488	0.0138	0.369	0.0166	28.2	29.4	38.9
9CS2.5x070	26.7	0.263	0.00967	0.206	0.0118	21.9	21.7	28.3
9CS2.5x065	27.5	0.208	0.00844	0.165	0.0103	19.9	19.3	25.1
9CS2.5x059	28.6	0.153	0.00705	0.123	0.00867	17.6	16.5	21.4
8CS4x105	29.5	1.02	0.0394	0.782	0.0102	36.3	45.7	61.3
8CS4x085	31.8	0.521	0.0273	0.415	0.00709	27.2	32.9	43.4
8CS4x070	34.2	0.283	0.0193	0.232	0.00504	21.1	24.2	31.6
8CS4x065	35.2	0.224	0.0169	0.185	0.00442	19.2	21.5	27.9
8CS4x059	36.5	0.165	0.0142	0.139	0.00372	17.0	18.4	23.8
8CS3.5x105	27.3	1.03	0.0327	0.782	0.0119	40.5	44.8	60.7
8CS3.5x085	29.4	0.526	0.0226	0.415	0.00830	30.5	32.5	43.1
8CS3.5x070	31.6	0.285	0.0159	0.232	0.00590	23.7	24.0	31.4
8CS3.5x065	32.6	0.226	0.0139	0.185	0.00517	21.5	21.3	27.8
8CS3.5x059	33.8	0.167	0.0117	0.139	0.00435	19.1	18.3	23.8
8CS2.5x105	22.2	1.06	0.0214	0.782	0.0179	46.9	41.2	56.6
8CS2.5x085	24.0	0.542	0.0146	0.415	0.0124	35.4	30.1	40.4
8CS2.5x070	25.9	0.292	0.0103	0.232	0.00878	27.5	22.4	29.6
8CS2.5x065	26.7	0.231	0.00895	0.185	0.00769	25.0	20.0	26.2
8CS2.5x059	27.7	0.170	0.00748	0.139	0.00646	22.2	17.2	22.4

Table III – 4

Distortional Buckling Properties
Axial Strength
C-Sections With Lips

$\Omega_c = 1.80$
$\phi_c = 0.85$



Section	Per Section 4.2(b)							
	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d ksi	P_n[1] $(L_m \geq L_{cr})$ (F_y=33 ksi) kips	P_n[1] $(L_m \geq L_{cr})$ (F_y=55 ksi) kips
8CS2x105	19.4	1.09	0.0170	0.782	0.0235	46.3	38.1	52.2
8CS2x085	21.0	0.555	0.0115	0.415	0.0163	35.0	27.8	37.3
8CS2x070	22.7	0.299	0.00799	0.232	0.0115	27.2	20.7	27.3
8CS2x065	23.3	0.236	0.00696	0.185	0.0100	24.8	18.4	24.2
8CS2x059	24.3	0.173	0.00580	0.139	0.00843	21.9	15.9	20.7
7CS4x105	28.5	1.14	0.0421	0.894	0.00728	41.2	45.1	61.1
7CS4x085	30.7	0.588	0.0292	0.474	0.00508	31.0	32.7	43.5
7CS4x070	33.1	0.320	0.0206	0.265	0.00361	24.1	24.2	31.7
7CS4x065	34.0	0.253	0.0180	0.212	0.00317	21.9	21.5	28.1
7CS4x059	35.3	0.187	0.0151	0.159	0.00266	19.4	18.5	24.0
7CS2.5x105	21.5	1.19	0.0229	0.894	0.0128	58.5	40.9	57.4
7CS2.5x085	23.2	0.610	0.0156	0.474	0.00888	44.2	30.4	41.4
7CS2.5x070	25.1	0.330	0.0110	0.265	0.00629	34.4	22.8	30.5
7CS2.5x065	25.8	0.261	0.00956	0.212	0.00551	31.4	20.4	27.1
7CS2.5x059	26.8	0.192	0.00800	0.159	0.00463	27.8	17.6	23.2
6CS4x105	27.5	1.31	0.0455	1.04	0.00495	46.7	44.1	60.5
6CS4x085	29.6	0.676	0.0315	0.553	0.00345	35.2	32.2	43.2
6CS4x070	31.8	0.368	0.0223	0.309	0.00246	27.4	23.9	31.6
6CS4x065	32.7	0.292	0.0195	0.247	0.00215	24.9	21.4	28.0
6CS4x059	34.0	0.216	0.0164	0.185	0.00181	22.1	18.4	24.0
6CS2.5x105	20.7	1.37	0.0247	1.04	0.00871	72.1	39.7	57.2
6CS2.5x085	22.4	0.700	0.0169	0.553	0.00604	54.6	30.0	41.7
6CS2.5x070	24.1	0.379	0.0118	0.309	0.00428	42.7	22.8	31.0
6CS2.5x065	24.8	0.300	0.0103	0.247	0.00375	38.9	20.5	27.6
6CS2.5x059	25.8	0.221	0.00864	0.185	0.00315	34.5	17.7	23.7
4CS4x105	24.8	1.89	0.0558	1.56	0.00180	60.1	41.2	58.0
4CS4x085	26.7	0.980	0.0386	0.830	0.00125	45.4	30.6	41.9
4CS4x070	28.7	0.536	0.0273	0.463	0.000892	35.5	23.1	30.9
4CS4x065	29.6	0.426	0.0239	0.371	0.000782	32.3	20.6	27.5
4CS4x059	30.7	0.315	0.0200	0.277	0.000658	28.6	17.8	23.6
4CS2.5x105	18.7	1.96	0.0303	1.56	0.00316	105	34.4	53.5
4CS2.5x085	20.2	1.01	0.0207	0.830	0.00219	80.3	27.3	40.0
4CS2.5x070	21.8	0.549	0.0145	0.463	0.00155	63.0	21.4	30.3
4CS2.5x065	22.4	0.436	0.0127	0.371	0.00136	57.6	19.4	27.1
4CS2.5x059	23.3	0.322	0.0106	0.277	0.00114	51.1	17.0	23.5
4CS2x105	17.5	1.96	0.0232	1.56	0.00361	131	31.6	51.4
4CS2x085	19.0	1.01	0.0156	0.830	0.00248	102	25.6	39.4
4CS2x070	20.6	0.547	0.0108	0.463	0.00174	80.8	20.6	30.3
4CS2x065	21.2	0.434	0.00935	0.371	0.00152	74.1	18.9	27.3
4CS2x059	22.1	0.321	0.00777	0.277	0.00127	66.2	16.7	23.8

Note:

1. Axial strengths given are nominal strengths. To obtain the available strength, these values must be modified by safety factors (ASD) or resistance factors (LRFD).

Table III – 5

Distortional Buckling Properties
SSMA Studs - Axial Strength
C-Sections With Lips

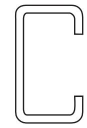
Ω_c = 1.80
ϕ_c = 0.85

Section	Per Section 4.2(b)							
	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d ksi	P_n[1] $_{(Lm \geq Lcr)}$ (F_y=33 ksi) kips	P_n[1] $_{(Lm \geq Lcr)}$ (F_y=50 ksi) kips
1200S250-97	19.7	0.775	0.0212	0.474	0.0749	13.0	28.8	35.2
1200S250-68	23.9	0.234	0.0105	0.163	0.0355	8.64	16.6	20.1
1200S250-54*	27.1	0.110	0.00664	0.0816	0.0219	6.69	11.6	14.0
1200S200-97	17.1	0.803	0.0158	0.474	0.0984	11.2	25.1	30.7
1200S200-68	20.9	0.241	0.00787	0.163	0.0464	7.45	14.5	17.5
1200S200-54*	23.7	0.112	0.00498	0.0816	0.0286	5.77	10.1	12.1
1200S162-97	12.9	0.938	0.0142	0.474	0.174	7.50	19.2	23.2
1200S162-68	15.8	0.273	0.00713	0.163	0.0815	4.92	10.9	13.1
1200S162-54*	17.9	0.124	0.00452	0.0816	0.0501	3.78	7.50	9.00
1000S250-97	18.8	0.898	0.0233	0.568	0.0475	20.7	31.9	39.6
1000S250-68	22.8	0.274	0.0115	0.196	0.0225	13.8	18.7	22.9
1000S250-54	25.9	0.129	0.00728	0.0980	0.0139	10.7	13.1	16.0
1000S250-43*	29.2	0.0618	0.00463	0.0496	0.00869	8.36	9.25	–
1000S200-97	16.4	0.929	0.0173	0.568	0.0624	18.8	28.5	35.3
1000S200-68	20.0	0.281	0.00862	0.196	0.0294	12.5	16.7	20.4
1000S200-54	22.7	0.131	0.00545	0.0980	0.0181	9.72	11.7	14.2
1000S200-43*	25.6	0.0629	0.00347	0.0496	0.0113	7.60	8.24	–
1000S162-97	12.3	1.08	0.0156	0.568	0.110	13.1	22.2	27.2
1000S162-68	15.1	0.316	0.00781	0.196	0.0517	8.60	12.8	15.5
1000S162-54	17.1	0.145	0.00495	0.0980	0.0317	6.62	8.91	10.8
1000S162-43*	19.4	0.0682	0.00316	0.0496	0.0197	5.15	6.23	–
800S250-97	17.8	1.08	0.0260	0.710	0.0272	33.7	34.2	43.4
800S250-68	21.6	0.332	0.0129	0.245	0.0129	22.4	20.5	25.5
800S250-54	24.5	0.157	0.00814	0.122	0.00796	17.3	14.6	18.0
800S250-43	27.6	0.0756	0.00517	0.0620	0.00498	13.6	10.4	–
800S200-97	15.5	1.11	0.0194	0.710	0.0357	33.1	31.5	39.9
800S200-68	18.9	0.340	0.00964	0.245	0.0168	22.1	18.9	23.5
800S200-54	21.4	0.160	0.00610	0.122	0.0104	17.1	13.4	16.6
800S200-43	24.2	0.0768	0.00388	0.0620	0.00648	13.4	9.56	–
800S200-33*	27.9	0.0331	0.00229	0.0280	0.00375	10.1	6.38	–
800S162-97	11.6	1.28	0.0174	0.710	0.0632	24.7	25.6	32.0
800S162-68	14.2	0.379	0.00873	0.245	0.0296	16.3	15.1	18.6
800S162-54	16.2	0.175	0.00554	0.122	0.0182	12.5	10.6	13.0
800S162-43	18.3	0.0828	0.00353	0.0620	0.0113	9.77	7.51	–
800S162-33*	21.1	0.0351	0.00208	0.0280	0.00652	7.33	4.97	–
800S137-97	8.59	1.56	0.0172	0.710	0.116	17.0	20.2	24.9
800S137-68	10.5	0.445	0.00873	0.245	0.0543	10.9	11.6	14.1
800S137-54	12.0	0.201	0.00556	0.122	0.0333	8.33	8.07	9.78
800S137-43	13.6	0.0931	0.00355	0.0620	0.0206	6.42	5.64	–
800S137-33*	15.7	0.0386	0.00210	0.0280	0.0118	4.78	3.70	–
600S250-97	16.5	1.37	0.0300	0.947	0.0132	53.6	34.4	45.1
600S250-68	20.1	0.427	0.0149	0.326	0.00627	35.6	21.3	27.1
600S250-54	22.8	0.203	0.00940	0.163	0.00388	27.6	15.4	19.3
600S250-43	25.7	0.0984	0.00597	0.0826	0.00242	21.6	11.1	–
600S200-97	14.4	1.41	0.0224	0.947	0.0174	59.4	32.3	42.8
600S200-68	17.6	0.436	0.0111	0.326	0.00820	39.5	20.3	26.0
600S200-54	19.9	0.206	0.00704	0.163	0.00506	30.6	14.7	18.6
600S200-43	22.5	0.0998	0.00448	0.0826	0.00315	23.9	10.6	–
600S200-33	25.9	0.0432	0.00264	0.0373	0.00183	18.0	7.18	–

Table III – 5

Distortional Buckling Properties
SSMA Studs - Axial Strength
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85



Section	Per Section 4.2(b)							
	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d ksi	P_n[1] $(L_m \geq L_{cr})$ (F_y=33 ksi) kips	P_n[1] $(L_m \geq L_{cr})$ (F_y=50 ksi) kips
600S162-97	10.8	1.60	0.0201	0.947	0.0308	50.1	27.8	36.2
600S162-68	13.3	0.481	0.0101	0.326	0.0144	33.0	17.1	21.7
600S162-54	15.1	0.224	0.00639	0.163	0.00885	25.4	12.3	15.4
600S162-43	17.1	0.107	0.00408	0.0826	0.00550	19.8	8.85	–
600S162-33	19.7	0.0455	0.00241	0.0373	0.00318	14.8	5.95	–
600S137-97	7.99	1.93	0.0198	0.947	0.0566	37.6	23.2	29.6
600S137-68	9.79	0.558	0.0101	0.326	0.0265	24.2	13.9	17.3
600S137-54	11.1	0.254	0.00642	0.163	0.0162	18.4	9.85	12.2
600S137-43	12.6	0.119	0.00410	0.0826	0.0100	14.2	6.99	–
600S137-33	14.6	0.0496	0.00242	0.0373	0.00577	10.6	4.64	–
550S162-68	13.0	0.518	0.0105	0.356	0.0116	39.5	17.4	22.3
550S162-54	14.8	0.241	0.00668	0.178	0.00712	30.4	12.6	16.0
550S162-43	16.7	0.115	0.00426	0.0901	0.00443	23.7	9.12	–
550S162-33	19.3	0.0493	0.00251	0.0407	0.00255	17.8	6.16	–
400S200-68	15.9	0.624	0.0136	0.490	0.00297	67.1	19.4	26.0
400S200-54	18.0	0.298	0.00862	0.245	0.00183	51.9	14.5	19.0
400S200-43	20.4	0.145	0.00549	0.124	0.00114	40.5	10.8	–
400S200-33	23.4	0.0632	0.00324	0.0559	0.000663	30.5	7.43	–
400S162-68	12.0	0.679	0.0123	0.490	0.00523	66.5	17.1	23.0
400S162-54	13.6	0.319	0.00783	0.245	0.00321	51.1	12.8	16.8
400S162-43	15.4	0.153	0.00499	0.124	0.00200	39.7	9.48	–
400S162-33	17.8	0.0660	0.00295	0.0559	0.00115	29.8	6.53	–
400S137-68	8.84	0.773	0.0123	0.490	0.00960	57.5	14.9	19.7
400S137-54	10.1	0.356	0.00786	0.245	0.00588	43.7	11.0	14.2
400S137-43	11.4	0.168	0.00502	0.124	0.00364	33.7	8.06	–
400S137-33	13.2	0.0710	0.00297	0.0559	0.00209	25.1	5.49	–
362S200-68	15.5	0.681	0.0143	0.540	0.00233	73.4	18.9	25.7
362S200-54	17.6	0.326	0.00906	0.270	0.00143	56.8	14.3	18.9
362S200-43	19.9	0.159	0.00577	0.137	0.000895	44.4	10.6	–
362S200-33	22.9	0.0693	0.00340	0.0617	0.000518	33.4	7.38	–
362S162-68	11.7	0.739	0.0130	0.540	0.00409	75.0	16.7	22.7
362S162-54	13.3	0.348	0.00823	0.270	0.00251	57.6	12.7	16.7
362S162-43	15.0	0.168	0.00524	0.137	0.00156	44.7	9.42	–
362S162-33	17.3	0.0723	0.00310	0.0617	0.000901	33.6	6.53	–
362S137-68	8.63	0.838	0.0130	0.540	0.00751	67.3	14.7	19.7
362S137-54	9.82	0.387	0.00826	0.270	0.00460	51.1	11.0	14.4
362S137-43	11.1	0.183	0.00527	0.137	0.00285	39.4	8.10	–
362S137-33	12.9	0.0775	0.00312	0.0617	0.00164	29.3	5.57	–
350S162-68	11.6	0.762	0.0132	0.560	0.00374	78.0	16.5	22.6
350S162-54	13.2	0.359	0.00837	0.280	0.00230	59.9	12.6	16.7
350S162-43	14.9	0.173	0.00534	0.142	0.00143	46.5	9.39	–
350S162-33	17.2	0.0747	0.00315	0.0639	0.000825	34.9	6.52	–
250S162-68	10.7	1.02	0.0156	0.783	0.00161	105	14.6	21.1
250S162-54	12.1	0.486	0.00990	0.392	0.000992	80.5	11.6	15.9
250S162-43	13.7	0.236	0.00631	0.198	0.000617	62.6	8.86	–
250S162-33	15.8	0.102	0.00373	0.0895	0.000356	47.0	6.29	–
250S137-68	7.86	1.14	0.0156	0.783	0.00296	104	12.9	18.5
250S137-54	8.95	0.532	0.00994	0.392	0.00182	78.6	10.2	13.9
250S137-43	10.2	0.254	0.00635	0.198	0.00112	60.5	7.76	–
250S137-33	11.7	0.109	0.00375	0.0895	0.000647	45.0	5.48	–

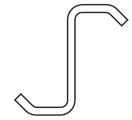
Note:

1. Axial strengths given are nominal strengths. To obtain the available strength, these values must be modified by safety factors (ASD) or resistance factors (LRFD).

Table III – 6

Distortional Buckling Properties
Axial Strength
Z-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85

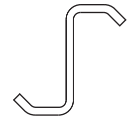


Section	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d ksi	P_n[1] $_{(L_m \geq L_{cr})}$ (F_y=33 ksi) kips	P_n[1] $_{(L_m \geq L_{cr})}$ (F_y=50 ksi) kips
	Per Section 4.2(b)							
12ZS3.25x105	26.8	0.765	0.0320	0.521	0.0415	17.5	39.1	50.6
12ZS3.25x085	29.4	0.384	0.0215	0.277	0.0279	13.4	27.8	35.6
12ZS3.25x070	32.0	0.205	0.0150	0.154	0.0195	10.4	20.2	25.7
12ZS2.75x105	24.2	0.786	0.0268	0.521	0.0512	16.8	36.4	47.0
12ZS2.75x085	26.5	0.393	0.0179	0.277	0.0343	12.8	25.8	33.1
12ZS2.75x070	28.8	0.209	0.0124	0.154	0.0239	10.0	18.8	23.9
12ZS2.25x105	21.3	0.815	0.0221	0.521	0.0661	15.1	32.8	42.3
12ZS2.25x085	23.4	0.405	0.0148	0.277	0.0442	11.6	23.2	29.7
12ZS2.25x070	25.4	0.215	0.0102	0.154	0.0308	9.02	16.8	21.4
10ZS3.25x105	25.6	0.893	0.0351	0.625	0.0263	24.7	41.3	54.2
10ZS3.25x085	28.1	0.450	0.0236	0.332	0.0177	18.9	29.6	38.4
10ZS3.25x070	30.5	0.241	0.0164	0.185	0.0123	14.8	21.7	27.9
10ZS3.25x065	31.5	0.190	0.0142	0.148	0.0107	13.5	19.2	24.7
10ZS3.25x059	33.0	0.140	0.0118	0.111	0.00894	12.1	16.5	21.1
10ZS2.75x105	23.1	0.915	0.0293	0.625	0.0324	24.9	39.1	51.4
10ZS2.75x085	25.4	0.459	0.0197	0.332	0.0218	19.1	28.1	36.4
10ZS2.75x070	27.5	0.245	0.0136	0.185	0.0152	15.0	20.6	26.4
10ZS2.75x065	28.5	0.193	0.0118	0.148	0.0132	13.7	18.2	23.4
10ZS2.75x059	29.7	0.142	0.00982	0.111	0.0110	12.2	15.6	20.0
10ZS2.25x105	20.3	0.947	0.0243	0.625	0.0419	23.8	36.0	47.2
10ZS2.25x085	22.3	0.473	0.0162	0.332	0.0280	18.2	25.8	33.4
10ZS2.25x070	24.3	0.252	0.0112	0.185	0.0195	14.2	18.9	24.3
10ZS2.25x065	25.1	0.198	0.00970	0.148	0.0170	13.0	16.8	21.5
10ZS2.25x059	26.2	0.145	0.00805	0.111	0.0141	11.6	14.3	18.3
9ZS2.25x105	19.8	1.03	0.0256	0.695	0.0322	29.9	37.3	49.5
9ZS2.25x085	21.8	0.517	0.0171	0.369	0.0215	22.9	26.9	35.2
9ZS2.25x070	23.7	0.276	0.0118	0.206	0.0150	18.0	19.8	25.7
9ZS2.25x065	24.5	0.217	0.0102	0.165	0.0130	16.4	17.6	22.7
9ZS2.25x059	25.6	0.159	0.00848	0.123	0.0108	14.6	15.1	19.4
8ZS3.25x105	24.2	1.08	0.0392	0.782	0.0150	34.3	42.1	56.3
8ZS3.25x085	26.6	0.546	0.0264	0.415	0.0101	26.3	30.5	40.2
8ZS3.25x070	28.9	0.294	0.0183	0.232	0.00706	20.7	22.6	29.4
8ZS3.25x065	29.8	0.232	0.0159	0.185	0.00615	18.9	20.1	26.1
8ZS3.25x059	31.2	0.171	0.0132	0.139	0.00512	16.9	17.3	22.3
8ZS2.75x105	21.8	1.11	0.0328	0.782	0.0186	36.8	40.5	54.4
8ZS2.75x085	24.0	0.557	0.0220	0.415	0.0125	28.2	29.5	39.0
8ZS2.75x070	26.1	0.299	0.0152	0.232	0.00869	22.2	21.8	28.5
8ZS2.75x065	26.9	0.236	0.0132	0.185	0.00756	20.3	19.5	25.3
8ZS2.75x059	28.1	0.173	0.0110	0.139	0.00628	18.1	16.7	21.7
8ZS2.25x105	19.2	1.14	0.0271	0.782	0.0240	37.6	38.1	51.3
8ZS2.25x085	21.1	0.572	0.0181	0.415	0.0160	28.9	27.8	36.8
8ZS2.25x070	23.0	0.306	0.0125	0.232	0.0112	22.7	20.6	26.9
8ZS2.25x065	23.7	0.241	0.0108	0.185	0.00971	20.7	18.3	23.9
8ZS2.25x059	24.8	0.177	0.00900	0.139	0.00806	18.5	15.8	20.5
7ZS2.25x105	18.6	1.28	0.0290	0.894	0.0172	47.0	38.3	52.5
7ZS2.25x085	20.4	0.643	0.0194	0.474	0.0115	36.2	28.2	37.9
7ZS2.25x070	22.2	0.344	0.0134	0.265	0.00799	28.5	21.1	27.9
7ZS2.25x065	23.0	0.271	0.0116	0.212	0.00695	26.1	18.9	24.8
7ZS2.25x059	24.0	0.199	0.00962	0.159	0.00577	23.2	16.3	21.3

Table III – 6

Distortional Buckling Properties
Axial Strength
Z-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85



Section	Per Section 4.2(b)							
	L_{cr} in.	$k_{\phi fe}$ kips	$\tilde{k}_{\phi fg}$ in.^{2}	$k_{\phi we}$ kips	$\tilde{k}_{\phi wg}$ in.^{2}	F_d ksi	P_n[1] $(L_m \geq L_{cr})$ (F_y=33 ksi) kips	P_n[1] $(L_m \geq L_{cr})$ (F_y=50 ksi) kips
6ZS2.25x105	17.9	1.46	0.0313	1.04	0.0117	58.2	37.7	52.8
6ZS2.25x085	19.7	0.735	0.0209	0.553	0.00781	44.8	28.2	38.5
6ZS2.25x070	21.4	0.395	0.0144	0.309	0.00544	35.4	21.3	28.5
6ZS2.25x065	22.1	0.312	0.0125	0.247	0.00473	32.4	19.1	25.4
6ZS2.25x059	23.1	0.229	0.0104	0.185	0.00393	28.9	16.5	21.8
4ZS2.25x070	19.3	0.568	0.0177	0.463	0.00197	52.5	20.3	28.2
4ZS2.25x065	20.0	0.450	0.0153	0.371	0.00172	48.1	18.3	25.2
4ZS2.25x059	20.9	0.331	0.0127	0.277	0.00143	43.0	16.0	21.8
3.5ZS1.5x070	11.8	0.721	0.0147	0.529	0.00357	68.3	16.3	23.3
3.5ZS1.5x065	12.1	0.568	0.0128	0.424	0.00312	62.3	14.8	20.9
3.5ZS1.5x059	12.7	0.416	0.0106	0.317	0.00260	55.3	13.0	18.2

Note:

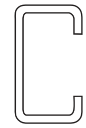
1. Axial strengths given are nominal strengths. To obtain the available strength, these values must be modified by safety factors (ASD) or resistance factors (LRFD).

1.4 Nominal Axial Strength Tables - Unbraced Columns

Table III - 7

Nominal Axial Strength, P_n, kips [1,2]
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85



Section	KL_x ft.	F_y = 33 ksi					F_y = 55 ksi				
		Bracing (KL_y = KL_t)					Bracing (KL_y= KL_t)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
12CS3.5x105	5.0	45.9†	45.8†	45.7†	45.3†	42.8†	63.0†	62.9†	62.6†	61.8†	57.0†
	10.0	45.2†	44.9†	44.4†	42.8†	32.5	61.6†	61.0†	60.1†	57.0†	38.4
	15.0	44.0†	43.3†	42.3	38.5	20.2	59.3†	58.0†	56.2†	48.6	20.2
	20.0	42.4	41.4	39.4	32.5	12.6	56.3†	54.2	50.2	38.4	12.6
	24.0	40.9	39.0	36.2	27.5		53.2	49.5	44.7	29.1	
	29.0	38.1	35.5	32.0	21.3		47.9	43.6	37.3	21.3	
	34.0	35.0	31.9	27.7	16.5		42.7	37.0	29.5	16.5	
	39.0	31.7	28.1	23.6	13.2		36.8	30.1	23.8	13.2	
12CS3.5x085	5.0	32.9†	32.8†	32.7†	32.5†	31.0†	44.6†	44.5†	44.3†	43.6†	39.7†
	10.0	32.4†	32.2†	32.0†	31.0†	24.4	43.4†	43.0†	42.2†	39.7†	27.9
	15.0	31.7†	31.3†	30.7†	27.9	15.1	41.6†	40.6†	39.0†	34.4	15.1
	20.0	30.7†	29.8	28.5	24.4	9.43	39.2†	37.5	35.2	27.9	9.43
	24.0	29.5	28.3	26.5	20.6		36.8	34.8	32.0	21.8	
	29.0	27.7	26.1	23.9	15.9		34.0	31.5	27.1	15.9	
	34.0	25.9	23.9	20.6	12.3		31.0	27.0	21.8	12.3	
	39.0	23.8	21.0	17.4	9.83		27.0	22.5	17.5	9.83	
12CS3.5x070	5.0	23.9†	23.8†	23.8†	23.5†	22.3†	32.8†	32.7†	32.5†	32.0†	29.3†
	10.0	23.5†	23.3†	23.1†	22.2†	18.2	32.0†	31.6†	31.1†	29.2†	20.3
	15.0	22.9†	22.5†	22.0†	20.3	11.6	30.6†	29.9†	28.7†	25.1	11.6
	20.0	22.1†	21.5	20.6	18.2	7.22	28.9†	27.6	25.8	20.3	7.22
	24.0	21.3	20.5	19.5	15.8		27.2	25.5	23.1	16.5	
	29.0	20.2	19.3	17.8	12.2		24.8	22.6	19.8	12.2	
	34.0	19.2	17.8	15.7	9.41		22.3	19.8	16.4	9.41	
	39.0	17.8	16.0	13.2	7.52		19.8	16.9	13.3	7.52	
10CS3.5x105	5.0	45.3†	45.2†	45.1†	44.6†	42.1	62.3†	62.1†	61.9†	61.0†	56.2
	9.0	44.5†	44.2	43.8	42.5	34.3	60.9†	60.3†	59.5†	56.9	42.0
	13.0	43.4	42.8	42.0	39.1	25.1	58.6	57.5	56.0	50.3	25.7
	17.0	41.9	41.0	39.5	34.4	16.7	55.8	53.9	51.0	42.1	16.7
	21.0	40.0	38.2	36.0	29.4		51.9	48.5	44.7	32.8	
	25.0	37.3	35.0	32.3	24.5		47.0	43.0	38.3	25.0	
	29.0	34.4	31.6	28.4	19.9		42.1	37.0	30.9	19.9	
	33.0	31.3	28.1	24.6	16.4		36.5	30.4	25.1	16.4	
10CS3.5x085	5.0	32.5†	32.5†	32.4†	32.1†	30.6	44.2†	44.0†	43.8†	43.1†	39.2
	9.0	32.1†	31.9†	31.7	30.8	25.3	43.0†	42.5†	41.9†	39.7	30.2
	13.0	31.4	31.0	30.6	28.3	18.4	41.2	40.3	39.1	35.2	18.7
	17.0	30.5	29.7	28.6	25.4	12.4	38.9	37.4	35.6	30.4	12.4
	21.0	29.0	27.8	26.5	21.9		36.2	34.3	32.1	24.2	
	25.0	27.3	25.9	24.2	18.1		33.5	31.2	27.8	18.3	
	29.0	25.6	23.8	21.3	14.5		30.7	27.1	23.1	14.5	
	33.0	23.7	21.1	18.3	11.9		26.8	22.8	18.7	11.9	

Table III - 7

Nominal Axial Strength, P_n, kips [1,2]
C-Sections With Lips

$\Omega_c = 1.80$
$\phi_c = 0.85$

Section	KL_x ft.	F_y = 33 ksi Bracing ($KL_y = KL_t$)					F_y = 55 ksi Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
10CS3.5x070	5.0	23.7†	23.6†	23.5†	23.3†	22.0	32.5†	32.4†	32.3†	31.7†	28.9
	9.0	23.3†	23.1	22.9	22.2	18.8	31.7†	31.3†	30.9†	29.3	21.6
	13.0	22.7	22.4	22.0	20.5	13.9	30.4†	29.7	28.8	25.8	14.1
	17.0	21.9	21.4	20.7	18.8	9.27	28.7	27.6	26.2	21.7	9.27
	21.0	21.0	20.3	19.5	16.5		26.8	25.2	23.3	17.8	
	25.0	20.0	19.2	18.0	13.7		24.5	22.5	20.2	13.9	
	29.0	19.0	17.8	16.1	11.0		22.1	19.8	17.2	11.0	
	33.0	17.7	16.1	14.1	8.93		19.8	17.2	14.3	8.93	
10CS3.5x065	5.0	21.1†	21.1†	21.0†	20.8†	19.6	28.9†	28.8†	28.7†	28.2†	25.7
	9.0	20.8†	20.6†	20.5	19.8	16.5	28.2†	27.9†	27.5†	26.1	19.2
	13.0	20.3	20.0	19.6	18.3	12.6	27.1†	26.5	25.7	23.0	12.7
	17.0	19.6	19.1	18.5	16.5	8.31	25.6	24.6	23.4	19.4	8.31
	21.0	18.7	18.1	17.2	14.7		23.8	22.5	20.8	15.8	
	25.0	17.8	16.9	15.9	12.4		21.9	20.1	17.9	12.5	
	29.0	16.7	15.8	14.4	9.85		19.7	17.6	15.4	9.85	
	33.0	15.7	14.4	12.7	8.02		17.5	15.4	12.9	8.02	
8CS3.5x105	4.0	44.5	44.5	44.4	44.0	42.1	61.6†	61.4†	61.2†	60.6	56.8
	7.0	43.9	43.6	43.3	42.3	36.5	60.3	59.8	59.2	57.3	46.1
	10.0	42.9	42.3	41.8	40.0	28.9	58.3	57.4	56.3	52.8	32.6
	14.0	41.1	40.2	39.0	34.9	19.5	54.9	53.1	50.7	43.4	19.5
	17.0	39.4	37.7	35.9	30.7	14.5	51.4	48.2	45.1	35.9	14.5
	20.0	36.9	34.8	32.6	26.4	11.3	46.9	43.1	39.8	28.0	11.3
	24.0	33.4	30.6	28.0	21.0		40.9	35.8	30.8	21.0	
	27.0	30.5	27.4	24.6	17.4		35.7	29.8	25.3	17.4	
8CS3.5x085	4.0	32.1	32.1	32.0	31.8	30.6	43.8†	43.6†	43.5†	43.0	39.9
	7.0	31.7	31.6	31.4	30.8	26.7	42.8	42.3	41.9	40.3	32.8
	10.0	31.1	30.8	30.5	29.0	21.6	41.2	40.4	39.5	36.6	24.0
	14.0	30.0	29.2	28.4	25.8	14.1	38.4	37.0	35.5	31.3	14.1
	17.0	28.7	27.6	26.5	23.1	10.5	35.9	34.2	32.4	26.2	10.5
	20.0	27.2	25.8	24.5	19.7	8.28	33.5	31.3	28.8	20.8	8.28
	24.0	25.0	23.3	21.2	15.5		30.0	26.5	23.2	15.5	
	27.0	23.3	20.8	18.5	12.8		26.5	22.6	19.1	12.8	
8CS3.5x070	4.0	23.4	23.4	23.3	23.2	22.2	32.3†	32.2†	32.1†	31.7†	29.5
	7.0	23.1	23.0	22.8	22.3	19.6	31.6†	31.3	30.9	29.8	23.9
	10.0	22.6	22.3	22.0	21.0	16.3	30.5	29.9	29.2	27.1	17.6
	14.0	21.7	21.2	20.6	19.1	10.6	28.5	27.4	26.3	22.6	10.6
	17.0	20.8	20.1	19.5	17.3	7.84	26.7	25.2	23.7	19.2	7.84
	20.0	19.9	19.1	18.3	15.2	6.13	24.6	22.8	20.9	15.8	6.13
	24.0	18.7	17.5	16.1	11.8		21.7	19.5	17.4	11.8	
	27.0	17.5	16.0	14.3	9.76		19.6	17.1	14.7	9.76	

Table III - 7

Nominal Axial Strength, P_n, kips [1,2]
C-Sections With Lips

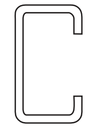
Ω_c = 1.80
ϕ_c = 0.85

Section	KL_x ft.	F_y = 33 ksi					F_y = 55 ksi				
		Bracing (KL_y = KL_t)					Bracing (KL_y= KL_t)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
8CS3.5x065	4.0	20.9	20.9	20.9	20.7	19.8	28.8†	28.7†	28.6†	28.2†	26.3
	7.0	20.6	20.5	20.4	19.9	17.4	28.1†	27.9†	27.6	26.6	21.3
	11.0	20.0	19.7	19.4	18.3	13.4	26.7	26.1	25.5	23.2	13.8
	14.0	19.4	18.9	18.4	16.8	9.54	25.4	24.4	23.4	20.2	9.54
	17.0	18.6	18.0	17.3	15.4	7.02	23.8	22.5	21.1	16.9	7.02
	21.0	17.4	16.5	15.7	12.9	5.09	21.4	19.6	17.8	13.2	5.09
	24.0	16.4	15.5	14.5	10.7		19.4	17.3	15.5	10.7	
	27.0	15.6	14.3	13.0	8.79		17.4	15.3	13.3	8.79	
8CS3.5x059	4.0	18.1	18.0	18.0	17.9	17.1	24.7†	24.7†	24.6†	24.3†	22.6
	7.0	17.8	17.7	17.6	17.2	15.0	24.2†	24.0†	23.7	22.8	18.3
	11.0	17.3	17.0	16.8	15.9	11.6	23.0	22.5	21.9	20.0	11.9
	14.0	16.7	16.4	15.9	14.6	8.31	21.9	21.1	20.2	17.4	8.31
	17.0	16.1	15.5	15.0	13.1	6.09	20.5	19.4	18.2	14.6	6.09
	21.0	15.1	14.3	13.5	11.2	4.39	18.4	16.9	15.4	11.4	4.39
	24.0	14.2	13.3	12.3	9.33		16.8	15.0	13.2	9.33	
	27.0	13.3	12.2	11.2	7.69		15.0	13.0	11.5	7.69	
8CS2.5x105	4.0	39.2	39.1	39.0	38.5	35.4	59.7†	59.4†	59.1†	57.8†	50.3
	7.0	38.4	38.1	37.7	36.3	28.2	57.7†	56.9†	55.9	52.3	34.3
	10.0	37.3	36.7	35.9	33.2	19.7	54.8	53.3	51.4	45.1	19.9
	13.0	35.7	34.8	33.6	29.5	12.5	51.0	48.8	45.9	37.1	12.5
	16.0	33.9	32.6	30.9	25.4		46.7	43.7	40.0	28.8	
	20.0	31.1	29.2	27.0	19.7		40.4	36.4	32.0	19.9	
	23.0	28.7	26.5	24.0	15.7		35.4	31.0	26.3	15.7	
	26.0	26.3	23.8	21.1	12.5		30.5	25.8	21.7	12.5	
8CS2.5x085	4.0	29.5	29.4	29.3	28.9	26.7	43.7†	43.5†	43.3†	42.6†	38.0
	7.0	28.9	28.7	28.4	27.2	21.2	42.6†	42.2†	41.6†	39.4	25.8
	10.0	28.1	27.6	27.0	24.9	14.9	41.1†	40.3	38.7	33.8	15.0
	13.0	26.9	26.2	25.2	22.1	9.70	38.6	36.8	34.5	27.6	9.70
	16.0	25.5	24.5	23.2	19.1		35.3	32.9	30.0	21.7	
	20.0	23.4	21.9	20.2	14.9		30.5	27.4	23.8	15.0	
	23.0	21.6	19.9	17.9	11.9		26.7	23.3	19.4	11.9	
	26.0	19.8	17.8	15.6	9.70		23.0	19.3	16.0	9.70	
8CS2.5x070	4.0	22.8†	22.7†	22.7†	22.3	20.6	31.2†	31.1†	30.9†	30.5†	28.2
	7.0	22.4	22.2	21.9	21.0	16.3	30.5†	30.3†	29.9†	28.8	19.9
	10.0	21.7	21.3	20.8	19.1	11.5	29.6†	29.1	28.5	25.9	11.6
	13.0	20.8	20.2	19.4	16.9	7.50	28.5	27.6	26.3	21.1	7.50
	16.0	19.7	18.9	17.8	14.5		26.8	25.5	23.1	16.4	
	20.0	18.0	16.9	15.5	11.4		23.6	21.1	18.2	11.5	
	23.0	16.7	15.3	13.7	9.19		20.7	17.9	14.8	9.19	
	26.0	15.3	13.7	11.9	7.50		17.8	14.9	12.1	7.50	

Table III - 7

Nominal Axial Strength, P_n, kips [1,2]
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85



Section	KL_x ft.	F_y = 33 ksi					F_y = 55 ksi				
		Bracing (KL_y = KL_t)					Bracing (KL_y= KL_t)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
8CS2.5x065	4.0	20.7†	20.6†	20.5†	20.3†	18.6	27.5†	27.4†	27.3†	26.8†	24.6
	7.0	20.3†	20.1†	19.9	19.0	14.8	26.8†	26.5†	26.2	25.1	18.1
	10.0	19.6	19.3	18.9	17.3	10.4	25.9	25.5	24.9	23.0	10.5
	13.0	18.8	18.3	17.6	15.3	6.78	24.9	24.2	23.3	19.1	6.78
	17.0	17.5	16.7	15.6	12.4		23.2	21.9	19.8	13.4	
	20.0	16.4	15.3	14.0	10.3		21.4	19.2	16.5	10.3	
	23.0	15.1	13.9	12.4	8.31		18.8	16.2	13.4	8.31	
	26.0	13.8	12.4	10.7	6.78		16.2	13.4	10.9	6.78	
8CS2.5x059	4.0	17.9†	17.9†	17.9†	17.7†	16.4	23.9†	23.8†	23.7†	23.2†	20.9
	7.0	17.7†	17.6†	17.4†	16.8	13.0	23.3†	23.0†	22.7†	21.5	15.9
	10.0	17.3†	17.0	16.6	15.2	9.11	22.3	21.9	21.2	19.3	9.21
	13.0	16.6	16.1	15.5	13.4	5.96	21.2	20.4	19.6	16.7	5.96
	17.0	15.4	14.7	13.7	10.8		19.5	18.6	17.4	11.7	
	20.0	14.4	13.5	12.3	8.92		18.3	16.9	14.5	8.99	
	23.0	13.3	12.2	10.8	7.21		16.5	14.3	11.7	7.21	
	26.0	12.2	10.9	9.39	5.96		14.2	11.8	9.56	5.96	
6CS2.5x105	3.0	37.9	37.8	37.7	37.4	35.4	58.4†	58.2†	57.9†	57.1	52.3
	5.0	37.2	37.0	36.8	35.9	31.2	56.8	56.2	55.5	53.3	42.3
	8.0	35.7	35.1	34.5	32.5	23.7	53.0	51.5	50.1	45.4	26.8
	10.0	34.4	33.5	32.6	29.8	18.6	49.7	47.6	45.6	39.3	19.0
	13.0	31.9	30.6	29.3	25.4	11.4	44.0	41.0	38.2	30.2	11.4
	15.0	30.1	28.4	26.9	22.4	8.56	39.8	36.3	33.1	24.5	8.56
	18.0	27.0	25.0	23.1	17.7		33.4	29.3	25.7	17.9	
	20.0	24.9	22.6	20.3	14.9		29.1	24.8	21.4	14.9	
6CS2.5x085	3.0	28.8	28.8	28.7	28.4	26.9	43.0†	42.8†	42.7†	42.2†	39.7
	5.0	28.4	28.2	28.0	27.3	23.6	42.1†	41.8†	41.4	40.3	31.8
	8.0	27.2	26.8	26.3	24.7	17.6	40.2	39.3	38.1	34.4	19.4
	10.0	26.2	25.5	24.8	22.6	13.8	37.9	36.3	34.7	29.6	13.8
	13.0	24.4	23.3	22.3	19.2	9.21	33.5	31.2	29.0	22.5	9.21
	15.0	23.0	21.7	20.5	16.8	6.96	30.4	27.7	25.1	18.1	6.96
	18.0	20.7	19.1	17.6	13.5		25.5	22.3	19.5	13.6	
	20.0	19.1	17.3	15.7	11.4		22.4	18.9	16.3	11.4	
6CS2.5x070	3.0	22.4	22.4	22.3	22.1	20.9	30.8	30.7	30.6	30.3	28.8
	5.0	22.1	21.9	21.8	21.2	18.3	30.3	30.1	29.9	29.1	24.6
	8.0	21.2	20.8	20.4	19.2	13.4	29.1	28.6	28.2	26.2	14.6
	10.0	20.4	19.9	19.3	17.5	10.3	28.1	27.4	26.4	22.9	10.3
	13.0	19.0	18.2	17.3	14.8	6.94	25.8	24.3	22.5	17.2	6.94
	15.0	17.9	16.9	15.9	12.9	5.54	23.7	21.5	19.4	13.8	5.54
	18.0	16.1	14.9	13.6	10.3		20.0	17.4	15.1	10.3	
	20.0	14.9	13.5	12.1	8.69		17.5	14.8	12.6	8.69	

Table III - 7

Nominal Axial Strength, P_n, kips [1,2]
C-Sections With Lips

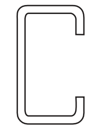
Ω_c = 1.80
ϕ_c = 0.85

Section	KL_x ft.	F_y = 33 ksi Bracing (KL_y = KL_t) Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	F_y = 55 ksi Bracing (KL_y= KL_t) Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
6CS2.5x065	3.0	20.4	20.3	20.3	20.1	19.0	27.2	27.1	27.0	26.7	25.2
	5.0	20.0	19.9	19.8	19.3	16.6	26.6	26.4	26.2	25.5	22.0
	8.0	19.2	18.9	18.6	17.4	12.1	25.5	25.1	24.6	23.2	13.1
	10.0	18.5	18.0	17.5	15.9	9.22	24.6	24.0	23.4	20.8	9.22
	13.0	17.2	16.5	15.7	13.4	6.18	23.0	21.8	20.4	15.6	6.18
	15.0	16.3	15.3	14.4	11.7	4.98	21.5	19.6	17.7	12.5	4.98
	18.0	14.7	13.5	12.4	9.25		18.1	15.8	13.6	9.26	
	20.0	13.5	12.2	11.0	7.82		15.9	13.4	11.4	7.82	
6CS2.5x059	3.0	17.7	17.7	17.7	17.5	16.8	23.7	23.6	23.5	23.2	21.6
	5.0	17.5	17.4	17.3	17.0	14.6	23.2	23.0	22.8	22.0	18.6
	8.0	17.0	16.7	16.4	15.4	10.6	22.0	21.5	21.0	19.5	11.4
	10.0	16.4	15.9	15.5	14.0	7.99	20.9	20.3	19.6	17.9	7.99
	13.0	15.2	14.6	13.9	11.8	5.33	19.3	18.5	17.7	13.7	5.33
	15.0	14.3	13.5	12.7	10.2	4.30	18.3	17.2	15.6	10.9	4.30
	18.0	12.9	11.9	10.9	8.08		16.0	13.9	12.0	8.08	
	20.0	12.0	10.8	9.69	6.81		14.0	11.8	10.0	6.81	
4CS2.5x105	2.0	34.1	34.0	33.9	33.7	32.3	55.5†	55.3†	55.1†	54.6†	51.3
	4.0	33.0	32.7	32.4	31.5	27.1	52.9	52.1	51.5	49.4	38.6
	6.0	31.3	30.6	30.1	28.4	21.0	48.8	47.2	45.8	41.5	25.2
	7.0	30.2	29.4	28.6	26.5	18.1	46.1	44.0	42.2	37.1	19.7
	9.0	27.7	26.5	25.4	22.6	13.2	40.0	37.0	34.6	28.4	13.2
	11.0	24.9	23.3	22.0	18.6	9.84	33.5	29.9	27.1	20.5	9.84
	12.0	23.4	21.6	20.2	16.7	8.75	30.3	26.4	23.6	17.5	8.75
	14.0	20.4	18.3	16.8	13.2	7.18	24.0	20.0	17.6	13.2	7.18
4CS2.5x085	2.0	27.3†	27.3	27.2	27.0	26.0	41.5†	41.4†	41.3†	40.9†	39.2
	4.0	26.5	26.3	26.1	25.5	21.8	40.1†	39.7	39.3	38.3	30.2
	6.0	25.3	24.8	24.4	22.9	16.5	37.9	36.7	35.7	32.6	19.3
	7.0	24.5	23.8	23.2	21.4	13.9	36.0	34.5	33.2	29.3	14.8
	9.0	22.5	21.5	20.6	18.1	9.64	31.8	29.6	27.7	22.6	9.64
	11.0	20.3	18.9	17.8	14.8	7.03	27.1	24.3	22.0	16.1	7.03
	12.0	19.1	17.6	16.3	13.2	6.18	24.8	21.5	19.1	13.7	6.18
	14.0	16.7	14.9	13.5	10.3	4.98	19.7	16.3	14.2	10.3	4.98
4CS2.5x070	2.0	21.6†	21.6†	21.5†	21.4	20.6	30.0	29.9	29.8	29.6	28.6
	4.0	21.0	20.8	20.7	20.1	17.4	29.1	28.9	28.7	28.0	23.6
	6.0	20.1	19.7	19.3	18.3	13.3	27.9	27.4	26.9	25.2	15.2
	7.0	19.5	18.9	18.5	17.1	11.1	27.1	26.4	25.6	23.1	11.6
	9.0	18.1	17.3	16.6	14.7	7.40	24.9	23.4	21.9	17.9	7.40
	11.0	16.5	15.4	14.5	12.0	5.28	21.6	19.4	17.6	13.0	5.28
	12.0	15.6	14.4	13.5	10.7	4.60	19.7	17.3	15.5	11.0	4.60
	14.0	13.8	12.3	11.1	8.24	3.64	16.1	13.5	11.6	8.24	3.64

Table III - 7

Nominal Axial Strength, P_n, kips [1,2]
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85



Section	KL_x ft.	F_y = 33 ksi					F_y = 55 ksi				
		Bracing (KL_y = KL_t)					Bracing (KL_y= KL_t)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
4CS2.5x065	2.0	19.7†	19.7†	19.6†	19.5†	18.8	26.6	26.5	26.4	26.2	25.1
	4.0	19.2	19.0	18.9	18.4	15.8	25.6	25.4	25.2	24.6	21.2
	6.0	18.3	18.0	17.6	16.7	12.1	24.5	24.0	23.6	22.4	13.7
	7.0	17.8	17.3	16.9	15.6	10.2	23.8	23.2	22.7	20.9	10.6
	9.0	16.5	15.8	15.2	13.4	6.71	22.2	21.1	20.0	16.3	6.71
	11.0	15.1	14.1	13.3	11.1	4.76	19.7	17.7	16.1	11.9	4.76
	12.0	14.3	13.2	12.3	9.88	4.13	18.1	15.9	14.2	10.2	4.13
	14.0	12.6	11.4	10.3	7.58	3.25	14.8	12.4	10.8	7.58	3.25
4CS2.5x059	2.0	17.2†	17.2†	17.2†	17.1†	16.6	23.2	23.1	23.1	22.9	21.8
	4.0	16.9	16.8	16.7	16.3	14.0	22.4	22.1	21.9	21.2	18.0
	6.0	16.3	15.9	15.7	14.8	10.7	21.1	20.6	20.1	18.9	12.0
	7.0	15.8	15.4	15.0	13.9	8.98	20.3	19.7	19.1	17.8	9.26
	9.0	14.7	14.0	13.5	11.9	5.94	18.7	18.0	17.4	14.4	5.94
	11.0	13.4	12.5	11.8	9.83	4.18	17.3	15.7	14.3	10.5	4.18
	12.0	12.7	11.7	10.9	8.82	3.61	16.1	14.1	12.6	9.04	3.61
	14.0	11.3	10.1	9.22	6.81	2.81	13.2	11.0	9.59	6.81	2.81

Note:

1. Axial strengths given are nominal strengths. To obtain the available strength, these values must be modified by safety factors (ASD) or resistance factors (LRFD).
2. The nominal strengths of members marked with the symbol † exceed the nominal distortional buckling strength of the member with no consideration of distortional buckling restraint from bracing or sheathing. In these cases, distortional buckling may control and the nominal strengths listed may be unconservative. See Table III-4 for distortional buckling strengths.

Table III - 8

Nominal Axial Strength, P_n, kips [1,2]
SSMA Studs
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85

Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing ($KL_y = KL_t$)					Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
1200S200-97	5.0	33.8†	33.3†	32.7†	31.2†	24.0	46.6†	45.6†	44.4†	41.2†	27.6
	9.0	33.2†	31.7†	30.0†	25.6†	11.8	45.5†	42.3†	38.8†	30.6	11.8
	14.0	32.1†	28.7†	25.1	17.2		43.2†	36.2†	29.6	17.3	
	18.0	31.0†	25.6†	20.6	11.8		40.8†	30.6	21.9	11.8	
	23.0	29.2†	21.5	15.1			37.2†	23.3	15.1		
	27.0	27.5†	18.1	11.8			34.0†	18.3	11.8		
	32.0	25.3†	14.1				29.8	14.1			
	36.0	23.3	11.8				26.5	11.8			
1200S200-68	5.0	20.6†	20.3†	20.0†	19.1†	14.8†	28.0†	27.6†	27.1†	25.5†	17.3
	9.0	20.3†	19.4†	18.4†	15.8†	7.38	27.5†	26.1†	24.1†	19.1†	7.38
	14.0	19.6†	17.6†	15.4†	10.8		26.5†	22.5†	18.5†	10.9	
	18.0	18.8†	15.8†	12.8	7.38		25.1†	19.1†	13.8	7.38	
	23.0	17.7†	13.3	9.44			22.8†	14.7	9.44		
	27.0	16.7†	11.2	7.38			20.8†	11.5	7.38		
	32.0	15.3†	8.84				18.2†	8.84			
	36.0	14.2	7.38				16.2	7.38			
1200S200-54*	5.0	15.1†	14.9†	14.6†	14.0†	10.9†	19.1†	18.9†	18.7†	18.0†	12.8†
	9.0	14.8†	14.2†	13.5†	11.6†	5.41	18.9†	18.2†	17.4†	14.1†	5.41
	14.0	14.3†	12.9†	11.3†	7.90		18.4†	16.6†	13.7†	8.02	
	18.0	13.8†	11.6†	9.38	5.41		17.8†	14.1†	10.2	5.41	
	23.0	13.0†	9.75	6.95			16.7†	10.9	6.95		
	27.0	12.2†	8.27	5.41			15.3†	8.50	5.41		
	32.0	11.2†	6.50	4.16			13.4†	6.50	4.16		
	36.0	10.3†	5.41				11.8	5.41			
1000S250-97	4.0	36.7†	36.6†	36.4†	35.8†	32.5†	49.0†	48.8†	48.5†	47.3†	42.0†
	8.0	36.0†	35.7†	34.9†	32.5†	22.1	47.8†	47.2†	45.8†	42.0†	23.6
	12.0	34.9†	34.1†	32.5†	27.6	12.3	45.8†	44.6†	42.0†	33.2	12.3
	16.0	33.4†	32.1†	29.4	22.1		43.4†	41.5†	36.5	23.6	
	20.0	31.5	29.8	25.8	16.5		40.6†	37.2	30.0	16.5	
	24.0	29.4	27.1	22.1	12.3		36.5	32.3	23.6	12.3	
	28.0	27.1	24.3	18.3			32.2	27.4	18.4		
	32.0	24.6	21.5	14.9			27.9	22.7	14.9		
1000S250-68	4.0	21.9†	21.8†	21.7†	21.4†	19.9†	27.9†	27.8†	27.7†	27.2†	24.8†
	8.0	21.5†	21.3†	21.0†	19.9†	13.9	27.3†	27.1†	26.5†	24.8†	15.0
	12.0	21.0†	20.6†	19.9†	17.3	7.84	26.5†	25.9†	24.8†	20.5	7.84
	16.0	20.2†	19.6†	18.3	13.9		25.4†	24.4†	22.0	15.0	
	20.0	19.4†	18.4	16.2	10.5		24.0†	22.1	19.0	10.5	
	24.0	18.2	16.7	13.9	7.84		21.9	19.7	15.0	7.84	
	28.0	16.8	14.9	11.6	6.13		19.8	16.8	11.7	6.13	
	32.0	15.2	13.2	9.46			17.4	13.8	9.46		

Table III - 8

Nominal Axial Strength, P_n, kips [1,2]
SSMA Studs
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85

Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing (KL_y = KL_t)					Bracing (KL_y= KL_t)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
1000S250-54	4.0	15.1†	15.1†	15.0†	14.8†	14.0†	19.5†	19.5†	19.3†	19.0†	17.1†
	8.0	14.9†	14.8†	14.6†	14.0†	10.3	19.1†	18.9†	18.5†	17.1†	11.2
	12.0	14.6†	14.4†	14.0†	12.6	5.80	18.4†	18.0†	17.1†	14.4	5.80
	16.0	14.2†	13.9†	13.2†	10.3		17.5†	16.7†	15.3	11.2	
	20.0	13.7†	13.2†	12.0	7.80		16.4†	15.3	13.6	7.80	
	24.0	13.1	12.2	10.3	5.80		15.1	13.9	11.2	5.80	
	28.0	12.3	10.9	8.61	4.53		14.0	12.3	8.73	4.53	
	32.0	11.2	9.61	7.02			12.6	10.1	7.02		
1000S250-43*	4.0	11.1†	11.1†	11.0†	10.9†	10.2†					
	8.0	10.9†	10.9†	10.7†	10.2†	7.68					
	12.0	10.7†	10.5†	10.2†	9.09	4.31					
	16.0	10.3†	10.0†	9.49†	7.68						
	20.0	9.91†	9.46†	8.65	5.82						
	24.0	9.40†	8.79	7.68	4.31						
	28.0	8.84	8.05	6.42	3.35						
	32.0	8.21	7.10	5.23							
1000S200-97	4.0	33.4†	33.2†	32.8†	31.9†	27.3	46.3†	45.8†	45.1†	43.1†	34.0
	8.0	32.7†	31.9†	30.6†	27.3	14.7	44.9†	43.1†	40.5†	34.0	14.7
	12.0	31.7†	29.9†	27.3	21.1		42.7†	39.0†	34.0	22.9	
	16.0	30.2†	27.3	23.2	14.7		39.8†	34.0	26.6	14.7	
	19.0	29.0†	25.1	20.0	11.2		37.2†	29.9	21.1	11.2	
	23.0	27.1	21.9	15.7			33.5	24.3	15.7		
	27.0	24.9	18.6	12.2			29.6	19.1	12.2		
	31.0	22.7	15.4	9.74			25.7	15.4	9.74		
1000S200-68	4.0	20.5†	20.4†	20.2†	19.6†	16.9†	27.9†	27.7†	27.4†	26.6†	21.3†
	8.0	20.1†	19.6†	18.8†	16.9†	9.30	27.3†	26.6†	25.2†	21.3†	9.30
	12.0	19.4†	18.4†	16.9†	13.1		26.4†	24.3†	21.3†	14.5	
	16.0	18.5†	16.9†	14.4	9.30		24.6†	21.3†	16.8	9.30	
	20.0	17.5†	15.1	11.8	6.56		22.5†	17.9	12.4	6.56	
	23.0	16.6	13.6	9.93			20.7†	15.4	9.95		
	27.0	15.3	11.7	7.74			18.3	12.2	7.74		
	31.0	13.9	9.78	6.23			15.8	9.78	6.23		
1000S200-54	4.0	15.0†	14.9†	14.8†	14.4†	12.4†	19.0†	19.0†	18.8†	18.4†	15.7†
	8.0	14.7†	14.4†	13.8†	12.4†	6.86	18.7†	18.4†	17.8†	15.7†	6.86
	12.0	14.2†	13.5†	12.4†	9.66		18.2†	17.5†	15.7†	10.8	
	16.0	13.6†	12.4†	10.6	6.86		17.6†	15.7†	12.4	6.86	
	20.0	12.8†	11.1	8.71	4.83		16.5†	13.2	9.19	4.83	
	23.0	12.1†	10.0	7.32			15.2†	11.4	7.34		
	27.0	11.1	8.59	5.70			13.4	9.01	5.70		
	31.0	10.1	7.21	4.58			11.6	7.22	4.58		

Table III - 8

Nominal Axial Strength, P_n, kips [1,2]
SSMA Studs
C-Sections With Lips

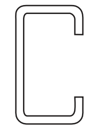
Ω_c = 1.80
ϕ_c = 0.85

Section	KL_x ft.	F_y = 33 ksi, Bracing ($KL_y = KL_t$) Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	F_y = 50 ksi, Bracing ($KL_y = KL_t$) Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
1000S200-43*	4.0	10.9†	10.8†	10.7†	10.5†	9.18†					
	8.0	10.7†	10.5†	10.2†	9.18†	5.09					
	12.0	10.4†	9.97†	9.18†	7.16						
	16.0	10.00†	9.18†	7.87	5.09						
	20.0	9.47†	8.21	6.46	3.55						
	23.0	8.96†	7.43	5.42							
	27.0	8.24	6.37	4.21							
	31.0	7.47	5.34	3.37							
800S200-97	4.0	32.6†	32.5†	32.2†	31.4	27.1	45.3†	45.1†	44.6†	42.8†	34.3
	7.0	31.9†	31.6†	30.8	28.4	18.2	43.9†	43.2†	41.6†	36.7	18.8
	10.0	30.9	30.3	28.8	24.3	10.2	41.9†	40.5†	37.5	29.1	10.2
	13.0	29.6	28.6	26.2	19.7		39.2	37.2	32.6	21.2	
	16.0	28.1	26.6	23.3	15.1		36.1	33.4	27.3	15.1	
	19.0	26.3	24.5	20.2	11.3		32.7	29.4	22.0	11.3	
	22.0	24.3	22.2	17.1	8.46		29.1	25.4	17.4	8.46	
	25.0	22.3	19.9	14.1			25.5	21.5	14.1		
800S200-68	4.0	20.2†	20.1†	20.0†	19.4†	16.9	27.6†	27.4†	27.2†	26.5†	21.6
	7.0	19.8†	19.5†	19.1†	17.7	11.5	26.9†	26.6†	26.0†	23.1	12.1
	10.0	19.1†	18.7	17.9	15.2	6.80	26.1†	25.2†	23.5†	18.4	6.80
	13.0	18.3	17.7	16.4	12.5		24.4†	23.1	20.5	13.6	
	16.0	17.4	16.4	14.6	9.72		22.5	20.7	17.3	9.74	
	19.0	16.3	15.1	12.8	7.39		20.4	18.1	14.1	7.39	
	22.0	15.1	13.7	10.9	5.83		18.1	15.6	11.2	5.83	
	25.0	13.8	12.2	9.12			15.9	13.1	9.12		
800S200-54	4.0	14.9†	14.8†	14.7†	14.3†	12.5	18.9†	18.8†	18.7†	18.3†	16.0
	7.0	14.5†	14.4†	14.1†	13.0	8.55	18.6†	18.4†	18.1†	16.9†	9.00
	10.0	14.1†	13.8†	13.2	11.2	5.06	18.1†	17.8†	17.1†	13.7	5.06
	13.0	13.5†	13.0	12.1	9.23		17.5†	16.8†	15.3	10.1	
	16.0	12.8	12.1	10.8	7.22		16.5	15.2	12.9	7.25	
	19.0	12.0	11.0	9.46	5.49		15.0	13.3	10.5	5.49	
	22.0	11.1	9.98	8.10	4.34		13.4	11.4	8.35	4.34	
	26.0	9.84	8.53	6.37			11.2	8.97	6.37		
800S200-43	4.0	10.8†	10.8†	10.7†	10.5†	9.29					
	7.0	10.6†	10.5†	10.4†	9.68†	6.38					
	10.0	10.3†	10.1†	9.79†	8.38	3.76					
	13.0	9.97†	9.64†	9.00	6.88						
	16.0	9.50	8.96	8.06	5.39						
	19.0	8.89	8.19	7.05	4.09						
	23.0	7.99	7.10	5.71	3.00						
	26.0	7.29	6.28	4.75							

Table III - 8

Nominal Axial Strength, P_n, kips [1,2]
SSMA Studs
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85

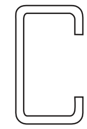


Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing (KL_y = KL_t)					Bracing (KL_y= KL_t)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
800S200-33*	4.0	7.04†	7.02†	6.99†	6.85†	6.16					
	7.0	6.93†	6.87†	6.77†	6.37	4.50					
	10.0	6.76†	6.64†	6.44†	5.68	2.66					
	13.0	6.54†	6.34	6.01	4.81						
	16.0	6.26	5.98	5.50	3.83						
	19.0	5.94	5.56	4.91	2.89						
	23.0	5.45	4.92	4.06	2.11						
	26.0	5.04	4.42	3.37							
800S162-97	4.0	24.1	23.9	23.5	22.4	17.4	33.7†	33.1†	32.3†	30.1	20.4
	7.0	23.6	22.9	21.7	18.8	8.88	32.6†	31.0	28.7	23.0	8.88
	10.0	22.8	21.4	19.3	14.4		30.9	28.0	23.9	15.3	
	13.0	21.8	19.5	16.4	10.0		28.8	24.3	18.7	10.0	
	16.0	20.6	17.4	13.4	7.13		26.4	20.4	13.8	7.13	
	19.0	19.2	15.1	10.5			23.7	16.6	10.5		
	22.0	17.7	12.9	8.23			21.0	13.1	8.23		
	25.0	16.1	10.7	6.66			18.2	10.7	6.66		
800S162-68	4.0	15.1†	15.0	14.7	14.1	11.1	21.3†	21.0†	20.5†	19.1†	13.2
	7.0	14.8	14.3	13.7	11.9	5.83	20.6†	19.7†	18.3	14.8	5.83
	10.0	14.3	13.5	12.2	9.25		19.5†	17.8	15.3	10.1	
	13.0	13.6	12.3	10.5	6.57		18.2	15.6	12.1	6.57	
	16.0	12.9	11.1	8.64	4.71		16.6	13.2	9.06	4.71	
	19.0	12.0	9.71	6.85			14.9	10.8	6.85		
	22.0	11.1	8.34	5.41			13.2	8.61	5.41		
	25.0	10.1	7.00	4.41			11.5	7.00	4.41		
800S162-54	4.0	11.2†	11.1†	10.9†	10.5	8.25	15.3†	15.2†	14.9†	14.2†	9.95
	7.0	11.0†	10.7†	10.2	8.89	4.38	14.9†	14.5†	13.7†	11.1	4.38
	10.0	10.6	10.0	9.09	6.92		14.4†	13.4†	11.5	7.60	
	13.0	10.1	9.19	7.82	4.94		13.6†	11.7	9.15	4.94	
	16.0	9.54	8.25	6.47	3.53		12.4	9.95	6.85	3.53	
	19.0	8.89	7.26	5.16			11.1	8.17	5.16		
	22.0	8.19	6.25	4.06			9.83	6.51	4.06		
	25.0	7.45	5.27	3.30			8.51	5.27	3.30		
800S162-43	4.0	8.42†	8.35†	8.22†	7.86†	6.21					
	7.0	8.23†	8.01†	7.64†	6.68	3.30					
	10.0	7.95†	7.52†	6.83	5.21						
	13.0	7.58†	6.91	5.88	3.73						
	16.0	7.14	6.21	4.87	2.65						
	19.0	6.65	5.46	3.89							
	22.0	6.12	4.70	3.05							
	25.0	5.56	3.97	2.48							

Table III - 8

Nominal Axial Strength, P_n, kips [1,2]
SSMA Studs
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85

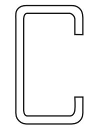


Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing (KL_y = KL_t)					Bracing (KL_y= KL_t)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
800S162-33*	4.0	5.81†	5.77†	5.70†	5.52†	4.47					
	7.0	5.71†	5.60†	5.41†	4.82	2.36					
	10.0	5.56†	5.34†	4.92	3.74						
	13.0	5.37†	4.97	4.23	2.67						
	16.0	5.11†	4.47	3.50	1.88						
	19.0	4.78	3.93	2.79							
	22.0	4.39	3.38	2.18							
	25.0	3.98	2.85	1.76							
600S200-97	3.0	31.4	31.3	31.2	30.8	28.6	44.2†	44.0†	43.8†	42.9†	38.3
	5.0	30.8	30.6	30.3	29.3	23.7	43.0†	42.5	41.9	39.8	29.0
	8.0	29.6	29.0	28.4	26.1	14.6	40.3	39.3	37.9	33.5	14.7
	10.0	28.4	27.6	26.7	23.7	9.40	38.0	36.5	34.6	28.8	9.40
	12.0	27.1	26.0	24.8	20.9		35.4	33.3	31.0	23.9	
	15.0	24.8	23.4	21.8	16.3		31.0	28.3	25.5	16.7	
	17.0	23.1	21.4	19.7	13.0		27.9	24.9	21.9	13.0	
	20.0	20.4	18.5	16.4	9.40		23.2	19.9	17.0	9.40	
600S200-68	3.0	19.8	19.7	19.7	19.4	18.0	27.2†	27.1†	27.0†	26.6†	24.2
	5.0	19.4	19.3	19.1	18.5	15.2	26.7†	26.5†	26.2†	25.1	18.7
	8.0	18.6	18.3	17.9	16.4	9.94	25.5	24.8	23.9	21.0	10.0
	10.0	17.9	17.4	16.8	14.8	6.90	24.1	23.0	21.8	17.8	6.90
	13.0	16.7	15.9	15.0	12.2		21.5	20.0	18.2	13.3	
	15.0	15.7	14.8	13.7	10.5		19.7	17.9	15.9	10.8	
	17.0	14.7	13.6	12.3	8.92		17.7	15.7	13.6	8.92	
	20.0	13.1	11.7	10.3	6.90		14.8	12.6	10.5	6.90	
600S200-54	3.0	14.7	14.6	14.6	14.4	13.4	18.7†	18.7†	18.6†	18.4	17.4
	5.0	14.4	14.3	14.2	13.7	11.2	18.5	18.3	18.2	17.7	13.7
	8.0	13.8	13.6	13.2	12.1	7.48	17.9	17.6	17.3	15.5	7.58
	10.0	13.3	12.9	12.4	10.9	5.23	17.3	16.8	16.2	13.1	5.23
	13.0	12.4	11.8	11.1	8.88		16.0	14.9	13.5	9.67	
	15.0	11.7	10.9	10.1	7.59		14.6	13.3	11.7	7.73	
	17.0	10.9	10.0	9.05	6.37		13.2	11.6	9.95	6.37	
	20.0	9.69	8.67	7.55	4.98		11.0	9.31	7.68	4.98	
600S200-43	3.0	10.7†	10.7†	10.7†	10.6	9.91					
	5.0	10.6	10.5	10.4	10.1	8.32					
	8.0	10.2	10.1	9.84	9.04	5.48					
	10.0	9.89	9.64	9.30	8.08	3.92					
	13.0	9.25	8.80	8.24	6.56						
	15.0	8.71	8.16	7.49	5.56						
	18.0	7.84	7.14	6.33	4.22						
	20.0	7.23	6.45	5.57	3.58						

Table III - 8

Nominal Axial Strength, P_n, kips [1,2]
SSMA Studs
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85

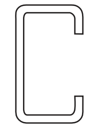


Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing (KL_y = KL_t)					Bracing (KL_y= KL_t)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
600S200-33	3.0	7.01	7.00	6.98	6.91	6.52					
	5.0	6.92	6.88	6.83	6.64	5.65					
	8.0	6.70	6.60	6.48	6.03	3.85					
	10.0	6.50	6.36	6.17	5.51	2.71					
	13.0	6.14	5.91	5.60	4.60						
	15.0	5.86	5.56	5.16	3.92						
	18.0	5.38	4.96	4.46	2.94						
	20.0	5.02	4.54	3.95	2.48						
600S162-97	3.0	23.3	23.2	23.1	22.5	19.7	32.9	32.7	32.4	31.2	25.5
	5.0	22.8	22.6	22.3	20.8	14.4	31.9	31.5	30.7	27.7	15.9
	7.0	22.2	21.8	21.1	18.5	8.65	30.6	29.8	28.4	23.2	8.65
	10.0	20.9	20.3	18.9	14.4		28.0	26.6	24.0	15.9	
	12.0	19.9	19.0	17.2	11.5		25.9	24.2	20.8	11.7	
	14.0	18.7	17.6	15.3	8.65		23.7	21.6	17.5	8.65	
	17.0	16.8	15.5	12.5	5.86		20.1	17.7	12.9	5.86	
	19.0	15.5	14.0	10.6			17.7	15.3	10.6		
600S162-68	3.0	14.8	14.8	14.7	14.4	12.7	21.0	20.9	20.7	20.0	16.6
	5.0	14.5	14.4	14.2	13.3	9.46	20.4	20.1	19.7	17.9	10.6
	8.0	13.9	13.6	13.1	11.2	4.87	19.1	18.4	17.4	13.6	4.87
	10.0	13.4	12.9	12.2	9.46		17.9	17.0	15.6	10.6	
	12.0	12.7	12.1	11.2	7.74		16.6	15.4	13.6	7.91	
	15.0	11.6	10.7	9.46	5.43		14.4	12.9	10.6	5.43	
	17.0	10.8	9.81	8.31	4.39		12.9	11.2	8.71	4.39	
	19.0	9.93	8.86	7.18			11.4	9.61	7.22		
600S162-54	3.0	11.1	11.0	11.0	10.7	9.50	15.1	15.1	15.0	14.7	12.5
	5.0	10.9	10.8	10.6	9.98	7.13	14.8	14.7	14.5	13.5	8.06
	8.0	10.4	10.1	9.77	8.38	3.72	14.2	13.8	13.0	10.3	3.72
	10.0	9.97	9.61	9.07	7.13		13.5	12.7	11.6	8.06	
	12.0	9.48	9.00	8.31	5.86		12.5	11.5	10.2	6.03	
	15.0	8.65	7.99	7.09	4.14		10.8	9.60	7.99	4.14	
	17.0	8.04	7.28	6.28	3.36		9.69	8.33	6.63	3.36	
	19.0	7.41	6.57	5.45			8.56	7.11	5.50		
600S162-43	3.0	8.37	8.34	8.29	8.12	7.19					
	5.0	8.21	8.13	8.01	7.55	5.40					
	8.0	7.85	7.66	7.37	6.34	2.82					
	10.0	7.53	7.26	6.84	5.40						
	12.0	7.16	6.79	6.24	4.45						
	15.0	6.52	6.01	5.29	3.15						
	17.0	6.06	5.47	4.66	2.55						
	19.0	5.59	4.92	4.05	2.11						

Table III - 8

Nominal Axial Strength, P_n, kips [1,2]
SSMA Studs
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85



Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing (KL_y = KL_t)					Bracing (KL_y= KL_t)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
600S162-33	3.0	5.79	5.77	5.75	5.66	5.16					
	5.0	5.71	5.67	5.60	5.38	3.91					
	8.0	5.52	5.42	5.26	4.59	2.03					
	10.0	5.36	5.19	4.93	3.91						
	12.0	5.13	4.90	4.50	3.22						
	15.0	4.71	4.33	3.80	2.27						
	17.0	4.38	3.93	3.33	1.83						
	20.0	3.85	3.33	2.66							
550S162-68	3.0	14.7	14.6	14.5	14.2	12.6	20.8	20.7	20.5	19.9	16.5
	5.0	14.3	14.2	14.0	13.2	9.44	20.1	19.8	19.4	17.8	10.7
	7.0	13.9	13.6	13.2	11.9	6.11	19.2	18.6	17.8	15.1	6.11
	9.0	13.3	12.9	12.3	10.3	3.88	18.0	17.1	16.0	12.1	3.88
	11.0	12.6	12.0	11.3	8.58		16.5	15.4	14.0	9.22	
	14.0	11.4	10.6	9.64	6.11		14.2	12.7	11.0	6.11	
	16.0	10.5	9.61	8.53	4.85		12.6	11.0	9.13	4.85	
	18.0	9.61	8.60	7.46	3.88		10.9	9.24	7.59	3.88	
550S162-54	3.0	11.0	11.0	10.9	10.7	9.49	15.1	15.0	14.9	14.6	12.5
	5.0	10.8	10.7	10.5	9.95	7.16	14.7	14.6	14.3	13.5	8.15
	7.0	10.4	10.2	9.93	8.96	4.69	14.2	14.0	13.4	11.5	4.69
	9.0	9.98	9.67	9.23	7.78	3.06	13.5	12.9	12.0	9.25	3.06
	11.0	9.46	9.03	8.43	6.53		12.5	11.6	10.5	7.08	
	14.0	8.56	7.95	7.16	4.69		10.7	9.55	8.15	4.69	
	16.0	7.90	7.18	6.30	3.74		9.47	8.19	6.70	3.74	
	18.0	7.22	6.41	5.47	3.06		8.25	6.89	5.54	3.06	
550S162-43	3.0	8.31	8.28	8.23	8.07	7.18					
	5.0	8.13	8.05	7.93	7.51	5.43					
	7.0	7.87	7.72	7.49	6.75	3.59					
	9.0	7.54	7.30	6.95	5.88	2.34					
	11.0	7.14	6.81	6.34	4.96						
	14.0	6.46	5.98	5.36	3.59						
	16.0	5.96	5.40	4.69	2.86						
	18.0	5.44	4.81	4.05	2.34						
550S162-33	3.0	5.78	5.76	5.74	5.65	5.17					
	5.0	5.68	5.64	5.58	5.37	3.95					
	7.0	5.55	5.47	5.35	4.90	2.61					
	9.0	5.38	5.23	5.02	4.25	1.69					
	11.0	5.14	4.93	4.60	3.58						
	14.0	4.69	4.33	3.86	2.61						
	16.0	4.32	3.90	3.37	2.07						
	18.0	3.94	3.47	2.89	1.69						

Table III - 8

Nominal Axial Strength, P_n, kips [1,2]
SSMA Studs
C-Sections With Lips

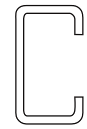
Ω_c = 1.80
ϕ_c = 0.85

Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing ($KL_y = KL_t$)					Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
400S162-68	2.0	14.0	13.9	13.9	13.8	13.0	20.2	20.1	20.0	19.7	18.0
	4.0	13.6	13.4	13.3	12.8	10.2	19.3	19.0	18.7	17.6	12.6
	5.0	13.3	13.1	12.8	12.1	8.65	18.6	18.2	17.7	16.2	10.0
	7.0	12.5	12.1	11.7	10.5	5.60	17.0	16.3	15.5	13.1	5.60
	9.0	11.5	11.0	10.4	8.65	3.39	15.1	14.0	12.9	9.99	3.39
	10.0	11.0	10.3	9.65	7.76		14.0	12.8	11.6	8.46	
	12.0	9.78	8.90	8.10	6.13		11.9	10.4	9.06	6.17	
	14.0	8.46	7.47	6.63	4.78		9.67	7.99	6.79	4.78	
400S162-54	2.0	10.7	10.7	10.6	10.5	9.90	14.8	14.7	14.7	14.5	13.7
	4.0	10.4	10.3	10.2	9.75	7.76	14.3	14.2	14.0	13.4	9.50
	5.0	10.1	9.99	9.81	9.22	6.57	14.0	13.8	13.5	12.3	7.38
	7.0	9.56	9.27	8.97	7.98	4.37	13.0	12.4	11.8	9.90	4.37
	9.0	8.84	8.41	7.96	6.64	2.82	11.6	10.7	9.87	7.49	2.82
	10.0	8.43	7.93	7.42	5.97	2.29	10.8	9.81	8.88	6.38	2.29
	12.0	7.57	6.93	6.33	4.71		9.14	8.01	6.97	4.71	
	14.0	6.65	5.92	5.25	3.59		7.52	6.29	5.35	3.59	
400S162-43	2.0	8.19	8.17	8.14	8.06	7.58					
	4.0	7.95	7.87	7.78	7.47	5.90					
	5.0	7.78	7.65	7.52	7.06	4.95					
	7.0	7.33	7.11	6.87	6.09	3.20					
	9.0	6.78	6.44	6.09	5.03	2.16					
	10.0	6.47	6.08	5.68	4.51	1.82					
	12.0	5.81	5.32	4.83	3.51						
	14.0	5.12	4.54	4.00	2.73						
400S162-33	2.0	5.71	5.70	5.69	5.64	5.40					
	4.0	5.59	5.55	5.50	5.34	4.30					
	5.0	5.50	5.43	5.36	5.10	3.57					
	7.0	5.27	5.13	4.98	4.44	2.25					
	9.0	4.93	4.71	4.45	3.65	1.50					
	10.0	4.73	4.44	4.14	3.26	1.27					
	12.0	4.25	3.88	3.52	2.51						
	14.0	3.74	3.31	2.90	1.93						
362S162-68	2.0	13.6	13.5	13.5	13.3	12.4	19.8	19.7	19.6	19.3	17.4
	3.0	13.4	13.3	13.1	12.8	11.0	19.4	19.2	18.9	18.2	14.7
	5.0	12.7	12.4	12.1	11.3	7.82	18.0	17.5	17.0	15.3	8.86
	6.0	12.2	11.8	11.5	10.4	6.46	17.1	16.4	15.7	13.6	6.67
	8.0	11.1	10.6	10.0	8.52	4.10	15.0	13.9	12.9	10.1	4.10
	9.0	10.5	9.84	9.24	7.61	3.24	13.8	12.5	11.4	8.50	3.24
	11.0	9.18	8.36	7.65	5.92		11.3	9.79	8.57	5.99	
	12.0	8.50	7.61	6.88	5.17		10.1	8.50	7.29	5.17	

Table III - 8

Nominal Axial Strength, P_n, kips [1,2]
SSMA Studs
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85

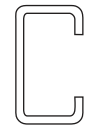


Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing (KL_y = KL_t)					Bracing (KL_y= KL_t)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
362S162-54	2.0	10.5	10.5	10.4	10.3	9.65	14.6	14.5	14.5	14.3	13.4
	3.0	10.3	10.3	10.2	9.95	8.60	14.4	14.2	14.1	13.8	11.2
	5.0	9.86	9.68	9.49	8.87	6.09	13.7	13.4	13.0	11.8	6.76
	6.0	9.54	9.29	9.03	8.21	4.87	13.2	12.6	12.1	10.5	4.94
	8.0	8.77	8.36	7.96	6.78	3.11	11.6	10.8	10.0	7.86	3.11
	9.0	8.33	7.85	7.38	6.02	2.61	10.7	9.80	8.94	6.64	2.61
	11.0	7.39	6.76	6.15	4.58		8.94	7.82	6.85	4.60	
	12.0	6.89	6.16	5.52	3.94		8.05	6.87	5.83	3.94	
362S162-43	2.0	8.09	8.06	8.04	7.95	7.43					
	4.0	7.80	7.71	7.61	7.28	5.64					
	5.0	7.60	7.46	7.31	6.82	4.64					
	7.0	7.08	6.82	6.56	5.76	2.88					
	8.0	6.77	6.45	6.14	5.20	2.30					
	10.0	6.08	5.65	5.23	4.08	1.61					
	11.0	5.72	5.23	4.77	3.56						
	13.0	4.96	4.38	3.87	2.64						
362S162-33	2.0	5.66	5.65	5.64	5.59	5.32					
	4.0	5.52	5.47	5.42	5.25	4.12					
	5.0	5.41	5.34	5.27	4.97	3.37					
	7.0	5.13	4.97	4.81	4.22	2.05					
	8.0	4.94	4.74	4.51	3.80	1.64					
	10.0	4.47	4.15	3.84	2.96	1.13					
	11.0	4.20	3.84	3.50	2.57						
	13.0	3.65	3.22	2.84	1.93						
350S162-68	2.0	13.3	13.2	13.2	13.0	12.1	19.6	19.5	19.4	19.1	17.2
	3.0	13.0	12.9	12.8	12.5	10.6	19.1	18.9	18.7	18.0	14.3
	5.0	12.3	12.0	11.8	10.9	7.53	17.7	17.1	16.6	14.9	8.45
	6.0	11.8	11.5	11.1	10.0	6.19	16.7	16.0	15.3	13.1	6.36
	8.0	10.7	10.1	9.62	8.15	4.03	14.4	13.3	12.3	9.54	4.03
	9.0	10.1	9.42	8.82	7.24	3.18	13.2	11.9	10.7	7.97	3.18
	11.0	8.74	7.91	7.23	5.57		10.6	9.11	7.94	5.61	
	12.0	8.05	7.16	6.46	4.84		9.36	7.83	6.73	4.84	
350S162-54	2.0	10.4	10.4	10.4	10.2	9.55	14.5	14.5	14.4	14.2	13.2
	3.0	10.3	10.2	10.1	9.85	8.46	14.3	14.1	14.0	13.7	11.0
	5.0	9.74	9.55	9.35	8.72	5.85	13.5	13.3	12.8	11.6	6.44
	6.0	9.40	9.13	8.87	8.03	4.66	13.0	12.4	11.9	10.2	4.70
	8.0	8.59	8.16	7.75	6.50	2.97	11.3	10.5	9.69	7.54	2.97
	9.0	8.13	7.62	7.13	5.72	2.50	10.4	9.44	8.58	6.21	2.50
	11.0	7.11	6.42	5.82	4.30		8.56	7.41	6.37	4.31	
	12.0	6.56	5.81	5.18	3.69		7.64	6.36	5.38	3.69	

Table III - 8

Nominal Axial Strength, P_n, kips [1,2]
SSMA Studs
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85

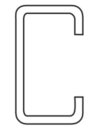


Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing (KL_y = KL_t)					Bracing (KL_y= KL_t)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
350S162-43	2.0	8.04	8.01	7.98	7.89	7.36					
	3.0	7.91	7.85	7.79	7.59	6.51					
	5.0	7.52	7.37	7.21	6.72	4.52					
	6.0	7.26	7.05	6.84	6.19	3.57					
	8.0	6.64	6.31	5.99	5.04	2.18					
	9.0	6.29	5.90	5.53	4.47	1.81					
	11.0	5.54	5.04	4.58	3.36						
	12.0	5.15	4.60	4.12	2.86						
350S162-33	2.0	5.66	5.64	5.63	5.58	5.30					
	3.0	5.59	5.56	5.53	5.43	4.79					
	5.0	5.39	5.31	5.23	4.93	3.29					
	6.0	5.26	5.13	5.01	4.56	2.56					
	8.0	4.88	4.66	4.42	3.71	1.58					
	9.0	4.65	4.36	4.08	3.27	1.28					
	11.0	4.10	3.72	3.38	2.45						
	12.0	3.81	3.40	3.03	2.11						
250S162-68	2.0	12.5	12.4	12.4	12.1	10.9	18.7	18.5	18.4	17.8	15.2
	3.0	12.1	11.9	11.8	11.3	9.12	17.8	17.4	17.0	16.0	11.6
	4.0	11.5	11.3	11.0	10.3	7.33	16.5	15.9	15.4	13.9	8.31
	5.0	10.9	10.4	10.1	9.11	5.77	15.1	14.2	13.5	11.6	5.89
	6.0	10.1	9.54	9.10	7.92	4.51	13.5	12.4	11.5	9.35	4.51
	7.0	9.25	8.57	8.05	6.76	3.67	11.8	10.5	9.59	7.34	3.67
	8.0	8.36	7.59	7.01	5.67	3.10	10.1	8.76	7.78	5.77	3.10
	9.0	7.46	6.61	6.01	4.69	2.70	8.53	7.09	6.21	4.69	2.70
250S162-54	2.0	10.1	10.1	10.00	9.83	8.83	14.1	14.0	14.0	13.7	12.1
	3.0	9.81	9.67	9.55	9.16	7.30	13.7	13.5	13.3	12.7	9.21
	4.0	9.37	9.13	8.92	8.31	5.72	13.1	12.7	12.3	11.1	6.37
	5.0	8.83	8.48	8.19	7.35	4.32	12.1	11.4	10.9	9.31	4.35
	6.0	8.21	7.75	7.38	6.36	3.24	10.9	10.1	9.36	7.47	3.24
	7.0	7.53	6.97	6.53	5.38	2.57	9.65	8.59	7.78	5.78	2.57
	8.0	6.82	6.17	5.68	4.46	2.13	8.30	7.14	6.29	4.51	2.13
	9.0	6.09	5.38	4.85	3.63	1.82	7.00	5.79	5.01	3.63	1.82
250S162-43	2.0	7.86	7.82	7.77	7.64	6.92					
	3.0	7.63	7.53	7.44	7.16	5.76					
	4.0	7.31	7.14	6.99	6.54	4.52					
	5.0	6.93	6.68	6.46	5.83	3.31					
	6.0	6.48	6.15	5.87	5.08	2.41					
	7.0	5.99	5.58	5.24	4.30	1.87					
	8.0	5.47	4.99	4.59	3.54	1.52					
	9.0	4.94	4.37	3.92	2.86	1.27					

Table III - 8

Nominal Axial Strength, P_n, kips [1,2]
SSMA Studs
C-Sections With Lips

Ω_c = 1.80
ϕ_c = 0.85



Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing ($KL_y = KL_t$)					Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
250S162-33	2.0	5.56	5.54	5.52	5.45	5.05					
	3.0	5.44	5.39	5.34	5.20	4.26					
	4.0	5.28	5.19	5.10	4.82	3.32					
	5.0	5.06	4.91	4.77	4.32	2.43					
	6.0	4.79	4.56	4.35	3.76	1.76					
	7.0	4.45	4.14	3.89	3.20	1.34					
	8.0	4.07	3.71	3.42	2.66	1.06					
	9.0	3.68	3.27	2.95	2.16	0.876					

Note:

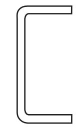
1. Axial strengths given are nominal strengths. To obtain the available strength, these values must be modified by of safety factors (ASD) or resistance factors (LRFD).

2. The nominal strengths of members marked with the symbol † exceed the nominal distortional buckling strength of the member with no consideration of distortional buckling restraint from bracing or sheathing. In these cases, distortional buckling may control and the nominal strengths listed may be unconservative. See Table III-5 for distortional buckling strengths.

Table III - 9

Nominal Axial Strength, P_n, kips [1]
SSMA Tracks
C-Sections Without Lips

Ω_c = 1.80
ϕ_c = 0.85

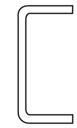


Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing (KL_y = KL_t)					Bracing (KL_y = KL_t)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
1200T200-97	5.0	29.0	28.4	27.8	26.2	18.8	37.3	36.2	35.1	32.2	19.7
	9.0	28.6	26.7	25.0	20.5		36.6	33.2	30.0	22.4	
	14.0	27.8	23.6	20.0	11.9		35.2	27.6	21.5	11.9	
	18.0	27.0	20.5	15.0			33.7	22.4	15.0		
	23.0	25.8	16.0	10.4			31.4	16.0	10.4		
	27.0	24.6	12.6				29.2	12.6			
	32.0	22.9	9.78				26.3	9.78			
	36.0	21.4					23.9				
1200T200-68	5.0	15.0	14.8	14.5	13.7	10.2	19.2	18.6	18.1	16.7	10.6
	9.0	14.9	14.0	13.1	11.0		18.8	17.1	15.6	11.9	
	14.0	14.5	12.5	10.7	6.98		18.1	14.5	11.5	6.98	
	18.0	14.1	11.0	8.49			17.3	11.9	8.49		
	23.0	13.4	8.91	6.19			16.2	8.95	6.19		
	27.0	12.9	7.31				15.1	7.31			
	32.0	12.0	5.81				13.7	5.81			
	36.0	11.3					12.5				
1200T200-54*	5.0	9.71	9.54	9.36	8.88	6.66	12.3	12.0	11.6	10.8	6.96
	9.0	9.60	9.04	8.52	7.17		12.1	11.0	10.1	7.79	
	14.0	9.36	8.10	7.00	4.65		11.6	9.37	7.52	4.65	
	18.0	9.10	7.17	5.61			11.2	7.79	5.61		
	23.0	8.71	5.88	4.16			10.4	5.90	4.16		
	27.0	8.33	4.87				9.77	4.87			
	32.0	7.81	3.95				8.87	3.95			
	36.0	7.36					8.10				
1000T200-97	4.0	28.6	28.3	27.9	27.0	22.3	37.0	36.4	35.8	34.0	25.6
	8.0	28.1	27.0	25.7	22.3	10.1	36.1	34.0	31.6	25.6	10.1
	12.0	27.4	24.9	22.3	15.7		34.8	30.2	25.6	15.8	
	16.0	26.4	22.3	18.1	10.1		33.0	25.6	18.9	10.1	
	20.0	25.2	19.3	13.4			30.7	20.6	13.4		
	23.0	24.2	16.6	10.8			28.9	16.8	10.8		
	27.0	22.6	13.2				26.2	13.2			
	31.0	20.9	10.6				23.3	10.6			
1000T200-68	4.0	14.9	14.8	14.6	14.1	11.9	19.0	18.8	18.5	17.6	13.6
	8.0	14.7	14.1	13.5	11.9	6.11	18.6	17.6	16.5	13.6	6.11
	12.0	14.3	13.2	11.9	8.84		17.9	15.8	13.6	8.89	
	16.0	13.8	11.9	9.90	6.11		17.0	13.6	10.3	6.11	
	20.0	13.2	10.4	7.77			15.9	11.1	7.77		
	23.0	12.7	9.24	6.47			15.0	9.37	6.47		
	27.0	12.0	7.65				13.7	7.65			
	31.0	11.1	6.38				12.3	6.38			

Table III - 9

Nominal Axial Strength, P_n, kips [1]
SSMA Tracks
C-Sections Without Lips

Ω_c = 1.80
ϕ_c = 0.85

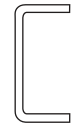


Section	KL_x ft.	F_y = 33 ksi, Bracing ($KL_y = KL_t$)					F_y = 50 ksi, Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
1000T200-54	4.0	9.65	9.56	9.46	9.17	7.77	12.2	12.1	11.9	11.4	8.84
	8.0	9.51	9.17	8.79	7.77	4.14	12.0	11.4	10.6	8.84	4.14
	12.0	9.28	8.56	7.77	5.86		11.6	10.2	8.84	5.89	
	16.0	8.98	7.77	6.52	4.14		11.0	8.84	6.79	4.14	
	19.0	8.70	7.09	5.52			10.5	7.70	5.52		
	23.0	8.27	6.11	4.36			9.71	6.20	4.36		
	27.0	7.79	5.11				8.87	5.11			
	31.0	7.26	4.30				7.98	4.30			
1000T200-43*	4.0	6.27	6.21	6.15	5.97	5.07					
	8.0	6.18	5.97	5.72	5.07	2.77					
	12.0	6.03	5.58	5.07	3.86						
	16.0	5.84	5.07	4.28	2.77						
	19.0	5.66	4.64	3.65							
	23.0	5.38	4.02	2.91							
	27.0	5.07	3.39								
	31.0	4.74	2.87								
800T200-97	4.0	27.9	27.7	27.4	26.5	22.3	36.2	35.9	35.3	33.7	26.0
	7.0	27.4	26.9	26.0	23.5	12.9	35.4	34.3	32.7	28.2	12.9
	10.0	26.8	25.7	23.9	19.5		34.1	32.1	28.9	21.3	
	13.0	25.9	24.1	21.4	14.5		32.5	29.3	24.5	14.6	
	16.0	24.8	22.3	18.5	10.4		30.5	26.0	19.7	10.4	
	19.0	23.6	20.2	15.1			28.3	22.5	15.2		
	23.0	21.7	17.1	11.2			25.0	17.7	11.2		
	26.0	20.1	14.5	8.95			22.4	14.6	8.95		
800T200-68	4.0	14.7	14.6	14.4	14.0	12.0	18.7	18.6	18.3	17.5	13.8
	7.0	14.4	14.2	13.7	12.6	7.61	18.3	17.9	17.0	14.9	7.61
	10.0	14.1	13.6	12.8	10.6		17.7	16.8	15.3	11.6	
	13.0	13.7	12.9	11.5	8.38		16.9	15.4	13.1	8.39	
	16.0	13.1	12.0	10.1	6.36		15.9	13.8	10.8	6.36	
	19.0	12.5	11.0	8.63			14.8	12.1	8.68		
	23.0	11.6	9.51	6.74			13.2	9.82	6.74		
	26.0	10.8	8.38	5.66			11.9	8.39	5.66		
800T200-54	4.0	9.52	9.47	9.37	9.12	7.84	12.1	12.0	11.8	11.3	9.04
	7.0	9.38	9.23	8.95	8.22	5.11	11.8	11.5	11.0	9.71	5.11
	10.0	9.17	8.86	8.34	6.99		11.4	10.9	9.92	7.61	
	13.0	8.89	8.40	7.57	5.59		10.9	10.0	8.57	5.60	
	16.0	8.55	7.84	6.69	4.32		10.3	9.04	7.12	4.32	
	19.0	8.16	7.21	5.75			9.60	7.98	5.79		
	22.0	7.72	6.54	4.82			8.84	6.87	4.82		
	26.0	7.08	5.59	3.90			7.76	5.60	3.90		

Table III - 9

Nominal Axial Strength, P_n, kips [1]
SSMA Tracks
C-Sections Without Lips

Ω_c = 1.80
ϕ_c = 0.85

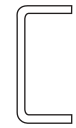


Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing ($KL_y = KL_t$)					Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
800T200-43	4.0	6.20	6.16	6.11	5.94	5.13					
	7.0	6.11	6.01	5.84	5.37	3.40					
	10.0	5.97	5.78	5.45	4.60						
	13.0	5.80	5.49	4.96	3.71						
	16.0	5.58	5.13	4.40	2.90						
	19.0	5.33	4.74	3.81							
	22.0	5.05	4.31	3.22							
	26.0	4.64	3.71	2.63							
800T200-33*	4.0	3.80	3.78	3.74	3.64	3.15					
	7.0	3.74	3.68	3.58	3.30	2.11					
	10.0	3.66	3.55	3.34	2.83						
	13.0	3.55	3.37	3.05	2.29						
	16.0	3.42	3.15	2.71	1.80						
	19.0	3.27	2.91	2.35	1.47						
	22.0	3.10	2.65	2.00							
	26.0	2.85	2.29	1.64							
800T125-97	3.0	24.9	24.3	23.7	22.1	15.2	33.8	32.5	31.3	28.2	15.9
	6.0	24.5	22.1	20.1	15.2		33.0	28.2	24.3	15.9	
	9.0	23.8	18.9	15.2	8.45		31.6	22.2	15.9	8.45	
	12.0	22.9	15.2	10.2			29.8	15.9	10.2		
	15.0	21.8	11.3	6.85			27.6	11.3	6.85		
	18.0	20.5	8.45				25.1	8.45			
	21.0	19.0					22.5				
	24.0	17.5					19.8				
800T125-68	3.0	14.0	13.7	13.4	12.7	8.98	18.1	17.6	17.0	15.6	9.51
	6.0	13.8	12.7	11.7	8.98		17.7	15.6	13.8	9.51	
	9.0	13.5	11.1	8.98	5.22		17.1	12.8	9.51	5.22	
	12.0	13.0	8.98	6.22			16.3	9.51	6.22		
	15.0	12.4	6.84	4.45			15.2	6.84	4.45		
	18.0	11.8	5.22				14.0	5.22			
	21.0	11.0					12.7				
	24.0	10.1					11.4				
800T125-54	3.0	9.21	9.03	8.86	8.38	6.20	11.8	11.5	11.1	10.3	6.53
	6.0	9.08	8.38	7.76	6.20		11.6	10.3	9.15	6.53	
	9.0	8.87	7.40	6.20	3.71		11.2	8.53	6.53	3.71	
	12.0	8.58	6.20	4.41			10.6	6.53	4.41		
	15.0	8.22	4.85	3.17			9.98	4.85	3.17		
	18.0	7.81	3.71				9.23	3.71			
	21.0	7.33					8.41				
	24.0	6.82					7.55				

Table III - 9

Nominal Axial Strength, P_n, kips [1]
SSMA Tracks
C-Sections Without Lips

$\Omega_c = 1.80$
$\phi_c = 0.85$

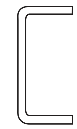


Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing ($KL_y = KL_t$)					Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
800T125-43	3.0	6.05	5.93	5.83	5.53	4.15					
	6.0	5.97	5.53	5.14	4.15						
	9.0	5.83	4.91	4.15	2.63						
	12.0	5.65	4.15	3.06							
	15.0	5.42	3.32	2.26							
	18.0	5.15	2.63								
	21.0	4.85									
	24.0	4.52									
800T125-33*	3.0	3.73	3.66	3.60	3.42	2.59					
	6.0	3.68	3.42	3.18	2.59						
	9.0	3.60	3.05	2.59	1.68						
	12.0	3.49	2.59	1.94							
	15.0	3.35	2.10	1.47							
	18.0	3.19	1.68								
	21.0	3.00									
	24.0	2.81									
600T200-97	3.0	26.8	26.7	26.6	26.2	24.0	35.2	35.0	34.8	34.0	29.8
	5.0	26.5	26.3	25.9	24.9	19.2	34.5	34.1	33.5	31.5	21.6
	8.0	25.6	25.1	24.4	21.8	9.82	32.9	32.0	30.6	26.0	9.82
	10.0	24.9	24.2	23.1	19.2		31.6	30.2	28.2	21.6	
	13.0	23.5	22.4	20.8	14.7		29.0	27.1	24.3	14.9	
	15.0	22.4	21.1	19.2	11.2		27.1	24.8	21.6	11.2	
	18.0	20.6	19.0	16.3	7.76		24.0	21.2	17.0	7.76	
	20.0	19.3	17.5	14.0			21.8	18.8	14.1		
600T200-68	3.0	14.3	14.3	14.2	14.0	12.9	18.4	18.3	18.2	17.8	15.8
	5.0	14.1	14.0	13.9	13.3	10.6	18.1	17.9	17.6	16.6	11.8
	8.0	13.7	13.5	13.1	11.9	6.51	17.3	16.8	16.1	13.9	6.51
	10.0	13.3	13.0	12.4	10.6		16.6	15.9	14.9	11.8	
	13.0	12.7	12.1	11.2	8.55		15.3	14.3	12.8	8.66	
	15.0	12.1	11.4	10.3	7.13		14.4	13.1	11.4	7.13	
	18.0	11.2	10.3	9.03	5.47		12.8	11.3	9.29	5.47	
	20.0	10.6	9.52	8.15			11.7	10.0	8.18		
600T200-54	3.0	9.34	9.32	9.28	9.15	8.47	11.9	11.9	11.8	11.6	10.3
	5.0	9.23	9.17	9.06	8.72	7.05	11.7	11.6	11.4	10.7	7.83
	8.0	8.97	8.81	8.55	7.77	4.46	11.2	10.9	10.5	9.05	4.46
	10.0	8.73	8.49	8.11	7.00	3.29	10.8	10.3	9.66	7.75	3.29
	13.0	8.30	7.92	7.35	5.75		9.99	9.31	8.33	5.83	
	15.0	7.96	7.48	6.79	4.85		9.39	8.56	7.39	4.85	
	18.0	7.39	6.77	5.91	3.81		8.40	7.37	6.04	3.81	
	20.0	6.98	6.27	5.33	3.29		7.71	6.56	5.33	3.29	

Table III - 9

Nominal Axial Strength, P_n, kips [1]
SSMA Tracks
C-Sections Without Lips

$\Omega_c = 1.80$
$\phi_c = 0.85$

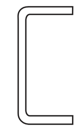


Section	KL_x ft.	$F_y = 33$ ksi					$F_y = 50$ ksi				
		Bracing ($KL_y = KL_t$)					Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
600T200-43	3.0	6.11	6.09	6.07	5.99	5.55					
	5.0	6.04	6.00	5.93	5.71	4.64					
	8.0	5.87	5.77	5.60	5.09	3.01					
	10.0	5.72	5.56	5.32	4.59	2.26					
	13.0	5.44	5.19	4.82	3.78						
	15.0	5.22	4.91	4.45	3.25						
	18.0	4.86	4.45	3.88	2.59						
	20.0	4.60	4.13	3.49	2.26						
600T200-33	3.0	3.76	3.75	3.73	3.68	3.41					
	5.0	3.71	3.69	3.65	3.51	2.86					
	8.0	3.61	3.55	3.45	3.13	1.89					
	10.0	3.52	3.42	3.27	2.82	1.44					
	13.0	3.35	3.20	2.97	2.32						
	15.0	3.22	3.03	2.74	1.99						
	18.0	3.00	2.74	2.38	1.61						
	20.0	2.84	2.55	2.14	1.43						
600T125-97	3.0	23.8	23.4	22.9	21.5	15.3	32.6	31.7	30.7	28.0	16.8
	5.0	23.4	22.2	20.9	17.6	6.54	31.7	29.4	26.9	20.7	6.54
	7.0	22.8	20.6	18.3	12.9		30.5	26.3	22.0	13.2	
	10.0	21.6	17.6	13.7	6.54		28.2	20.7	14.3	6.54	
	12.0	20.6	15.3	10.2			26.2	16.8	10.2		
	14.0	19.5	12.9	7.51			24.1	13.2	7.51		
	17.0	17.6	9.05				20.8	9.05			
	19.0	16.3	7.25				18.5	7.25			
600T125-68	3.0	13.6	13.4	13.1	12.5	9.26	17.7	17.3	16.8	15.6	10.2
	5.0	13.4	12.8	12.2	10.6	4.64	17.3	16.3	15.1	12.2	4.64
	7.0	13.1	12.1	10.9	7.93		16.7	14.9	12.9	8.14	
	10.0	12.5	10.6	8.38	4.64		15.6	12.2	8.77	4.64	
	12.0	12.0	9.26	6.62			14.7	10.2	6.62		
	14.0	11.4	7.93	5.19			13.7	8.14	5.19		
	17.0	10.4	6.02				12.0	6.02			
	19.0	9.66	5.04				10.9	5.04			
600T125-54	3.0	9.00	8.88	8.73	8.33	6.41	11.6	11.3	11.1	10.3	6.98
	5.0	8.88	8.55	8.16	7.14	3.39	11.3	10.7	10.0	8.20	3.39
	7.0	8.69	8.07	7.36	5.62		11.0	9.85	8.59	5.76	
	10.0	8.31	7.14	5.89	3.39		10.3	8.20	6.15	3.39	
	12.0	8.00	6.41	4.78			9.71	6.98	4.78		
	14.0	7.64	5.62	3.77			9.07	5.76	3.77		
	17.0	7.03	4.36				8.02	4.36			
	19.0	6.59	3.67				7.28	3.67			

Table III - 9

Nominal Axial Strength, P_n, kips [1]
SSMA Tracks
C-Sections Without Lips

Ω_c = 1.80
ϕ_c = 0.85

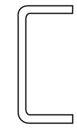


Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing ($KL_y = KL_t$)					Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
600T125-43	3.0	5.94	5.86	5.77	5.52	4.30					
	5.0	5.86	5.66	5.41	4.76	2.45					
	7.0	5.74	5.36	4.91	3.81						
	10.0	5.50	4.76	3.98	2.45						
	12.0	5.30	4.30	3.30							
	14.0	5.07	3.81	2.70							
	17.0	4.68	3.05								
	19.0	4.40	2.63								
600T125-33	3.0	3.68	3.64	3.58	3.43	2.70					
	5.0	3.63	3.51	3.36	2.97	1.58					
	7.0	3.56	3.33	3.06	2.40						
	10.0	3.41	2.97	2.50	1.58						
	12.0	3.29	2.70	2.09							
	14.0	3.15	2.40	1.73							
	17.0	2.91	1.94								
	19.0	2.74	1.69								
550T150-68	3.0	13.8	13.7	13.6	13.1	11.1	17.8	17.7	17.4	16.7	13.1
	5.0	13.5	13.4	13.0	11.9	7.24	17.4	17.1	16.4	14.4	7.24
	7.0	13.2	12.9	12.2	10.3	3.99	16.8	16.2	14.9	11.6	3.99
	9.0	12.8	12.3	11.1	8.34		16.1	15.1	13.1	8.53	
	11.0	12.3	11.5	9.95	6.21		15.1	13.8	11.1	6.21	
	14.0	11.4	10.3	7.98	3.99		13.5	11.6	8.09	3.99	
	16.0	10.7	9.32	6.52			12.4	10.0	6.52		
	18.0	10.0	8.34	5.38			11.1	8.53	5.38		
550T150-54	3.0	9.08	9.04	8.95	8.70	7.44	11.6	11.6	11.4	10.9	8.67
	5.0	8.94	8.83	8.59	7.93	5.08	11.4	11.2	10.7	9.53	5.08
	7.0	8.74	8.53	8.08	6.90	3.08	11.0	10.6	9.80	7.74	3.08
	9.0	8.48	8.14	7.44	5.70		10.5	9.91	8.67	5.84	
	11.0	8.16	7.69	6.71	4.46		9.93	9.11	7.42	4.46	
	14.0	7.59	6.90	5.49	3.08		8.92	7.74	5.57	3.08	
	16.0	7.16	6.31	4.65			8.18	6.78	4.65		
	18.0	6.70	5.70	3.95			7.40	5.84	3.95		
550T150-43	3.0	5.98	5.95	5.90	5.74	4.95					
	5.0	5.89	5.81	5.67	5.26	3.45					
	7.0	5.76	5.62	5.35	4.60	2.22					
	9.0	5.59	5.37	4.95	3.85						
	11.0	5.38	5.08	4.48	3.06						
	14.0	5.02	4.58	3.72	2.22						
	16.0	4.74	4.22	3.18							
	18.0	4.45	3.85	2.73							

Table III - 9

Nominal Axial Strength, P_n, kips [1]
SSMA Tracks
C-Sections Without Lips

Ω_c = 1.80
ϕ_c = 0.85



Section	KL_x ft.	F_y = 33 ksi, Bracing ($KL_y = KL_t$)					F_y = 50 ksi, Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
550T150-33	3.0	3.70	3.68	3.65	3.55	3.07					
	5.0	3.64	3.60	3.51	3.26	2.17					
	7.0	3.56	3.48	3.32	2.86	1.43					
	9.0	3.46	3.33	3.07	2.41						
	11.0	3.34	3.15	2.79	1.93						
	14.0	3.11	2.84	2.33	1.43						
	16.0	2.95	2.62	2.01							
	18.0	2.77	2.39	1.74							
400T125-68	2.0	12.9	12.8	12.7	12.5	11.2	17.0	16.9	16.7	16.3	13.9
	4.0	12.6	12.4	12.1	11.2	6.69	16.4	16.1	15.6	13.9	6.79
	5.0	12.3	12.1	11.7	10.3	4.34	16.0	15.6	14.8	12.3	4.34
	7.0	11.8	11.4	10.6	8.01		15.0	14.2	12.9	8.63	
	9.0	11.1	10.5	9.21	5.36		13.7	12.6	10.6	5.36	
	10.0	10.7	9.98	8.41	4.34		12.9	11.7	9.30	4.34	
	12.0	9.75	8.83	6.69	3.02		11.4	9.99	6.79	3.02	
	14.0	8.66	7.66	4.99			9.71	8.07	4.99		
400T125-54	2.0	8.64	8.61	8.57	8.42	7.62	11.2	11.2	11.1	10.8	9.33
	4.0	8.46	8.35	8.19	7.62	5.02	10.9	10.7	10.4	9.33	5.08
	5.0	8.32	8.17	7.92	7.06	3.51	10.6	10.3	9.87	8.34	3.51
	7.0	7.96	7.69	7.26	5.74		9.96	9.45	8.68	6.13	
	9.0	7.51	7.10	6.43	4.22		9.14	8.41	7.26	4.22	
	10.0	7.25	6.78	5.97	3.51		8.68	7.85	6.51	3.51	
	12.0	6.68	6.09	5.02	2.44		7.69	6.70	5.08	2.44	
	14.0	6.06	5.37	3.97			6.64	5.56	3.97		
400T125-43	2.0	5.76	5.74	5.71	5.62	5.12					
	4.0	5.64	5.58	5.47	5.12	3.47					
	5.0	5.55	5.45	5.29	4.77	2.59					
	7.0	5.33	5.14	4.86	3.93						
	9.0	5.03	4.76	4.35	3.00						
	10.0	4.87	4.55	4.08	2.59						
	12.0	4.50	4.10	3.47	1.95						
	14.0	4.10	3.62	2.85							
400T125-33	2.0	3.60	3.59	3.57	3.52	3.21					
	4.0	3.53	3.49	3.42	3.21	2.22					
	5.0	3.47	3.41	3.31	3.00	1.69					
	7.0	3.34	3.22	3.04	2.50						
	9.0	3.16	2.99	2.72	1.94						
	10.0	3.06	2.86	2.55	1.69						
	12.0	2.83	2.57	2.20	1.32						
	14.0	2.59	2.28	1.85							

Table III - 9

Nominal Axial Strength, P_n, kips [1]
SSMA Tracks
C-Sections Without Lips

Ω_c = 1.80
ϕ_c = 0.85

Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing (KL_y = KL_t)					Bracing (KL_y = KL_t)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
362T125-68	2.0	12.6	12.6	12.5	12.3	11.0	16.7	16.6	16.5	16.0	13.8
	4.0	12.3	12.1	11.8	11.0	6.47	16.0	15.7	15.3	13.8	6.61
	5.0	12.0	11.8	11.4	10.2	4.23	15.6	15.1	14.4	12.2	4.23
	7.0	11.4	10.9	10.3	7.80		14.4	13.6	12.5	8.63	
	8.0	11.0	10.5	9.76	6.47		13.7	12.7	11.5	6.61	
	10.0	10.1	9.34	8.26	4.23		12.1	10.9	9.28	4.23	
	11.0	9.55	8.72	7.35	3.50		11.2	9.95	7.87	3.50	
	13.0	8.33	7.28	5.63			9.38	7.75	5.63		
362T125-54	2.0	8.52	8.49	8.44	8.30	7.54	11.1	11.0	11.0	10.7	9.28
	4.0	8.30	8.19	8.03	7.52	4.99	10.7	10.5	10.2	9.23	5.09
	5.0	8.14	7.98	7.74	7.00	3.41	10.4	10.1	9.63	8.31	3.41
	7.0	7.72	7.44	7.03	5.70		9.61	9.10	8.37	6.14	
	8.0	7.47	7.13	6.64	4.99		9.16	8.54	7.69	5.09	
	10.0	6.90	6.43	5.81	3.41		8.15	7.33	6.32	3.41	
	11.0	6.58	6.05	5.39	2.82		7.60	6.71	5.64	2.82	
	13.0	5.90	5.28	4.50			6.47	5.47	4.50		
362T125-43	2.0	5.70	5.68	5.65	5.55	5.07					
	3.0	5.64	5.60	5.53	5.33	4.35					
	5.0	5.45	5.35	5.19	4.70	2.59					
	6.0	5.33	5.19	4.97	4.33	1.91					
	8.0	5.03	4.79	4.46	3.47						
	9.0	4.85	4.57	4.18	3.01						
	11.0	4.46	4.09	3.61	2.25						
	12.0	4.24	3.84	3.32	1.91						
362T125-33	2.0	3.57	3.56	3.54	3.48	3.18					
	3.0	3.53	3.51	3.47	3.35	2.75					
	5.0	3.42	3.36	3.26	2.95	1.70					
	6.0	3.35	3.26	3.12	2.72	1.33					
	8.0	3.16	3.02	2.80	2.21						
	9.0	3.05	2.88	2.63	1.95						
	11.0	2.81	2.58	2.26	1.50						
	12.0	2.68	2.42	2.08	1.33						
350T125-68	2.0	12.5	12.5	12.4	12.2	11.0	16.6	16.5	16.4	15.9	13.7
	3.0	12.4	12.3	12.1	11.6	9.03	16.3	16.1	15.8	14.9	10.5
	5.0	11.9	11.6	11.3	10.1	4.19	15.4	14.9	14.3	12.2	4.19
	6.0	11.6	11.2	10.8	9.03	2.91	14.8	14.2	13.3	10.5	2.91
	8.0	10.8	10.3	9.58	6.39		13.4	12.5	11.3	6.54	
	9.0	10.3	9.72	8.86	5.17		12.6	11.5	10.3	5.17	
	11.0	9.27	8.34	7.25	3.46		10.8	9.56	7.79	3.46	
	12.0	8.60	7.60	6.39	2.91		9.92	8.38	6.54	2.91	

Table III - 9

Nominal Axial Strength, P_n, kips [1]
SSMA Tracks
C-Sections Without Lips

Ω_c = 1.80
ϕ_c = 0.85

Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing (KL_y = KL_t)					Bracing (KL_y = KL_t)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
350T125-54	2.0	8.47	8.44	8.39	8.25	7.50	11.0	11.0	10.9	10.6	9.24
	3.0	8.37	8.31	8.21	7.90	6.36	10.9	10.7	10.6	9.96	7.24
	5.0	8.07	7.91	7.66	6.94	3.38	10.3	9.98	9.54	8.24	3.38
	6.0	7.86	7.64	7.32	6.36	2.35	9.91	9.50	8.91	7.24	2.35
	8.0	7.37	7.01	6.53	4.98		9.00	8.38	7.55	5.08	
	9.0	7.08	6.66	6.11	4.17		8.49	7.76	6.84	4.17	
	11.0	6.43	5.90	5.26	2.79		7.38	6.49	5.47	2.79	
	12.0	6.09	5.50	4.84	2.35		6.79	5.84	4.90	2.35	
350T125-43	2.0	5.67	5.65	5.62	5.53	5.04					
	3.0	5.61	5.57	5.51	5.30	4.34					
	5.0	5.41	5.31	5.15	4.66	2.59					
	6.0	5.28	5.14	4.92	4.29	1.89					
	8.0	4.96	4.73	4.40	3.47						
	9.0	4.78	4.50	4.11	3.01						
	11.0	4.36	4.00	3.53	2.25						
	12.0	4.14	3.73	3.24	1.89						
350T125-33	2.0	3.56	3.55	3.53	3.47	3.17					
	3.0	3.52	3.50	3.46	3.33	2.73					
	5.0	3.40	3.34	3.24	2.93	1.70					
	6.0	3.32	3.23	3.10	2.69	1.33					
	8.0	3.13	2.98	2.77	2.18						
	9.0	3.01	2.84	2.59	1.93						
	11.0	2.76	2.53	2.22	1.50						
	12.0	2.63	2.36	2.03	1.33						
250T125-68	2.0	11.0	10.9	10.8	10.6	9.22	15.4	15.2	15.1	14.6	12.0
	3.0	10.7	10.6	10.4	9.82	7.67	14.9	14.6	14.3	13.2	9.20
	4.0	10.3	10.1	9.80	8.97	5.70	14.2	13.7	13.1	11.5	5.96
	5.0	9.86	9.50	9.12	8.08	3.81	13.3	12.5	11.8	9.85	3.81
	6.0	9.31	8.84	8.38	7.13	2.65	12.2	11.2	10.4	8.33	2.65
	7.0	8.70	8.12	7.54	6.26		11.0	9.92	8.99	6.86	
	8.0	8.02	7.29	6.70	5.49		9.76	8.59	7.61	5.67	
	9.0	7.25	6.47	5.89	4.70		8.52	7.21	6.26	4.71	
250T125-54	2.0	7.89	7.85	7.80	7.62	6.73	10.4	10.3	10.2	9.90	8.30
	3.0	7.73	7.63	7.52	7.16	5.47	10.1	9.92	9.71	9.05	6.33
	4.0	7.50	7.34	7.16	6.55	4.38	9.66	9.37	9.04	8.03	4.54
	5.0	7.21	6.98	6.69	5.83	3.08	9.13	8.71	8.25	6.96	3.08
	6.0	6.87	6.50	6.12	5.13	2.14	8.52	7.96	7.39	5.75	2.14
	7.0	6.42	5.97	5.53	4.48		7.84	7.15	6.43	4.68	
	8.0	5.93	5.41	4.94	3.88		7.10	6.22	5.42	3.90	
	9.0	5.43	4.85	4.36	3.29		6.25	5.27	4.52	3.29	

Table III - 9

Nominal Axial Strength, P_n, kips [1]
SSMA Tracks
C-Sections Without Lips

Ω_c = 1.80
ϕ_c = 0.85

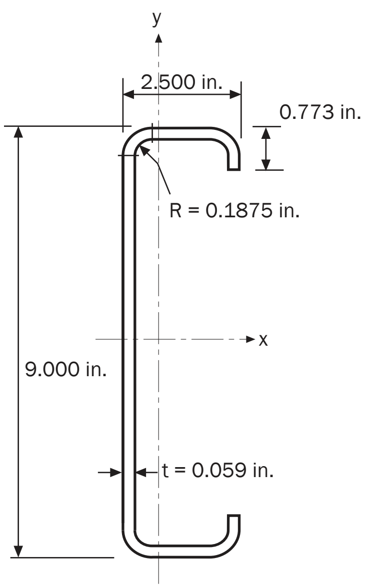
Section	KL_x ft.	F_y = 33 ksi					F_y = 50 ksi				
		Bracing ($KL_y = KL_t$)					Bracing ($KL_y = KL_t$)				
		Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None	Cont.	1/4 Pt.	1/3 Pt.	1/2 Pt.	None
250T125-43	2.0	5.38	5.35	5.31	5.20	4.61					
	3.0	5.27	5.21	5.13	4.89	3.82					
	4.0	5.12	5.01	4.89	4.51	2.99					
	5.0	4.93	4.78	4.60	4.08	2.28					
	6.0	4.72	4.50	4.28	3.63	1.73					
	7.0	4.47	4.20	3.92	3.14						
	8.0	4.19	3.87	3.55	2.65						
	9.0	3.90	3.53	3.14	2.26						
250T125-33	2.0	3.42	3.40	3.38	3.31	2.94					
	3.0	3.35	3.31	3.27	3.12	2.42					
	4.0	3.26	3.20	3.12	2.88	1.89					
	5.0	3.15	3.05	2.94	2.60	1.44					
	6.0	3.01	2.88	2.74	2.31	1.14					
	7.0	2.86	2.70	2.52	2.01						
	8.0	2.70	2.49	2.28	1.72						
	9.0	2.52	2.28	2.05	1.48						

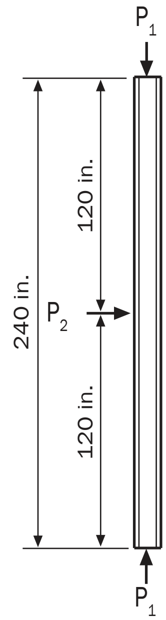
Note:

1. Axial strengths given are nominal strengths. To obtain the available strength, these values must be modified by safety factors (ASD) or resistance factors (LRFD)

SECTION 2 - EXAMPLE PROBLEMS

Example III-1: Braced C-Section With Lips - Bending And Compression





P_1 = 0.50 kips (dead)
= 2.0 kips (live)
P_2 = 0.10 kips (dead)
= 0.50 kips (live)

A_g = 0.881 in.^{2}
I_x = 10.3 in.^{4}
r_x = 3.42 in.

Given:

1. Steel: F_y = 55 ksi
2. Section: 9CS2.5x059 as shown above
3. Section simply supported at ends
4. Section fully braced against lateral-torsional, flexural-torsional and distortional buckling
5. K_x = 1.0; L_x = 240 in.

Required:

Verify the combined bending and compression strength of the section using ASD and LRFD methods with ASCE/SEI 7-05 load combinations.

Solution:

1. Refer to Example I-1 for derivation of geometric parameters.
2. Refer to Example I-8 for calculation of effective section properties.

Nominal flexural strength, M_n (Section C3.1)

Since the section is not subject to lateral-torsional or distortional buckling,

$$M_n = S_eF_y \quad (Eq.\ C3.1.1\text{-}1)$$

where S_e is calculated with the extreme fibers at F_y

From Table II-1 or Example I-8, S_e = 1.89 in.^{3}

$$M_n = (1.89)(55) \quad (Eq.\ C3.1.1\text{-}1)$$

$$= 104 \text{ kip-in.}$$

Nominal axial strength, P_n (Section C4.1)

Since the member can only buckle perpendicular to the x-axis,

$$F_e = \frac{\pi^2 E}{\left(KL_x/r_x\right)^2} \qquad (Eq.\ C4.1.1\text{-}1)$$

$$= \frac{\pi^2\left(29500\right)}{\left[(1.0)(240)/(3.42)\right]^2} \qquad (Eq.\ C4.1.1\text{-}1)$$

$$= 59.12 \text{ ksi}$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C4.1\text{-}4)$$

$$= \sqrt{\frac{55}{59.12}}$$

$$= 0.965 < 1.5, \text{ therefore}$$

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \qquad (Eq.\ C4.1\text{-}2)$$

$$= \left(0.658^{0.965^2}\right)55 = 37.25 \text{ ksi}$$

In Example I-8, the effective area at f = 37.25 ksi was calculated as:

$$A_e = 0.515 \text{ in.}^2$$

$$P_n = A_e F_n \qquad (Eq.\ C4.1\text{-}1)$$

$$= (0.515)(37.25) = 19.2 \text{ kips}$$

Required strength

$$M = \frac{P_2 L}{4}$$

$$M_{dead} = \frac{(0.10)(240)}{4} = 6.0 \text{ kip-in.}$$

$$M_{live} = \frac{(0.50)(240)}{4} = 30.0 \text{ kip-in.}$$

ASD

$$M_x = M_{dead} + M_{live} = 6.0 + 30.0 = 36.0 \text{ kip-in.}$$

$$P = P_{dead} + P_{live} = 0.5 + 2.0 = 2.5 \text{ kips}$$

LRFD

$$M_{ux} = 1.2M_{dead} + 1.6M_{live} = (1.2)(6.0) + (1.6)(30.0) = 55.2 \text{ kip-in.}$$

$$P_u = 1.2P_{dead} + 1.6P_{live} = (1.2)(0.5) + (1.6)(2.0) = 3.80 \text{ kips}$$

Combined compression and bending - ASD (Section C5.2.1)

$$\frac{\Omega_c P}{P_n} = \frac{(1.80)(2.5)}{19.2} = 0.234 > 0.15, \text{ therefore use Equations C5.2.1-1 and C5.2.1-2.}$$

$$C_{mx} = 1.0$$

$$P_{Ex} = \frac{\pi^2 EI_x}{(K_x L_x)^2} \quad (Eq.\ C5.2.1\text{-}6)$$

$$= \frac{\pi^2 (29500)(10.3)}{\left[(1.0)(240)\right]^2} = 52.1 \text{ kips}$$

$$\alpha_x = 1 - \frac{\Omega_c P}{P_{Ex}} > 0 \quad (Eq.\ C5.2.1\text{-}4)$$

$$= 1 - \frac{(1.80)(2.5)}{52.1} = 0.914$$

M_y = 0.0

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{mx} M_x}{M_{nx}\alpha_x} + \frac{\Omega_b C_{my} M_y}{M_{ny}\alpha_y} \le 1.0 \quad (Eq.\ C5.2.1\text{-}1)$$

$$\frac{(1.80)(2.5)}{19.2} + \frac{(1.67)(1.0)(36.0)}{(104)(0.914)} = 0.867 < 1.0 \quad \text{OK}$$

$$\frac{\Omega_c P}{P_{no}} + \frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} \le 1.0 \quad (Eq.\ C5.2.1\text{-}2)$$

From Table III-1, P_{no} = 24.3 kips

$$\frac{(1.80)(2.5)}{24.3} + \frac{(1.67)(36.0)}{104} = 0.763 < 1.0 \quad \text{OK} \quad (Eq.\ C5.2.1\text{-}2)$$

Combined compression and bending - LRFD (Section C5.2.2)

$\overline{P}$ = P_u = 3.80 kips

$\overline{M}_x$ = M_{ux} = 55.2 kip-in.

$$\frac{\overline{P}}{\phi_c P_n} = \frac{3.80}{(0.85)(19.2)} = 0.233 > 0.15, \text{ therefore use Equations C5.2.2-1 and C5.2.2-2}$$

C_{mx} = 1.0

P_{Ex} = 52.1 kips (computed in part 4 above)

$$\alpha_x = 1 - \frac{\overline{P}}{P_{Ex}} > 0 \quad (Eq.\ C5.2.2\text{-}4)$$

$$\alpha_x = 1 - \frac{3.80}{52.1} = 0.927$$

M_y = 0.0

$$\frac{\overline{P}}{\phi_c P_n} + \frac{C_{mx}\overline{M}_x}{\phi_b M_{nx}\alpha_x} + \frac{C_{my}\overline{M}_y}{\phi_b M_{ny}\alpha_y} \le 1.0 \quad (Eq.\ C5.2.2\text{-}1)$$

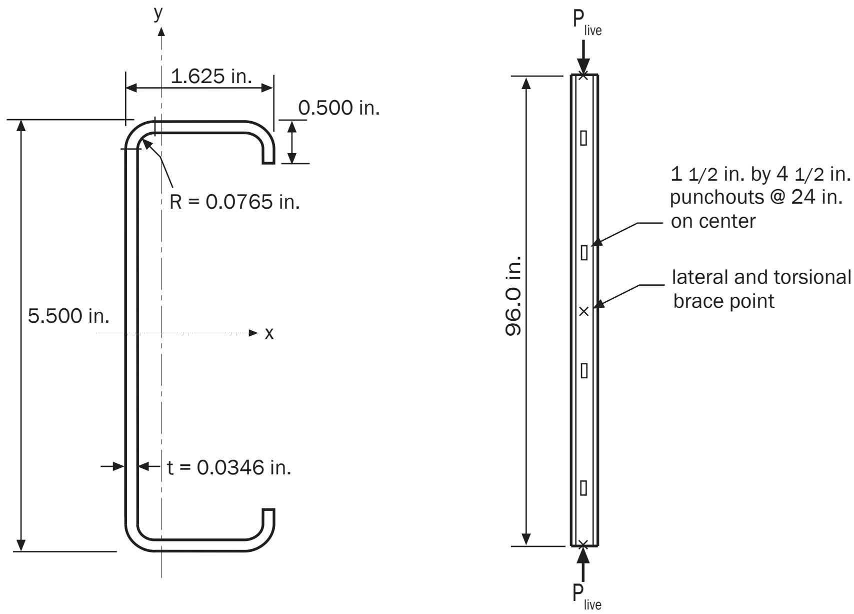
$$\frac{3.80}{(0.85)(19.2)} + \frac{(1.0)(55.2)}{(0.95)(104)(0.927)} = 0.836 < 1.0 \quad \text{OK}$$

$$\frac{\overline{P}}{\phi_c P_{no}} + \frac{\overline{M}_x}{\phi_b M_{nx}} + \frac{\overline{M}_y}{\phi_b M_{ny}} \le 1.0 \quad (Eq.\ C5.2.2\text{-}2)$$

from Table III-1, P_{no} = 24.3 kips

$$\frac{3.80}{(0.85)(24.3)} + \frac{55.2}{(0.95)(104)} = 0.743 < 1.0 \quad \text{OK}$$

Example III-2: C-Section With Lips With Holes – Compression



$A = 0.327\ \text{in.}^2$
$r_x = 2.11$ in.
$r_y = 0.589$ in.
$J = 0.000130\ \text{in.}^4$
$C_w = 0.713\ \text{in.}^6$
$x_o = -1.11$ in.
$r_o = 2.46$ in.

Given:

1. Steel: F_y = 33 ksi
2. Section: 550S162-33 as shown above
3. Concentrically loaded
4. Braced against buckling about the x-axis at ends only
5. Braced against buckling about the y-axis and for torsion at ends and mid-span
6. Braced against distortional buckling continuously
7. $K_x = K_y = K_t = 1.0$

Required:

1. Permitted applied load, P_{live}, using ASD and LRFD methods using the "all steel design" approach as described in Section D4.1 of the *Specification*.
2. Required brace strength and stiffness for the mid-span brace

Solution:

Axial strength

a) Check flexural buckling (Section C4.1).

$$\frac{K_x L_x}{r_x} = \frac{(1.0)(96.0)}{2.11} = 45.5$$

$$\frac{K_y L_y}{r_y} = \frac{(1.0)(48.0)}{0.589} = 81.5$$

Since $\frac{K_y L_y}{r_y} > \frac{K_x L_x}{r_x}$, Euler buckling about the y-axis will control.

$$F_e = \frac{\pi^2 E}{\left(K_y L_y / r_y\right)^2} \qquad (Eq.\ C4.1.1\text{-}1)$$

$$= \frac{\pi^2 (29500)}{\left[(1.0)(48.0)/(0.589)\right]^2}$$

$= 43.84$ ksi

b) Check flexural–torsional buckling (Section C4.1.2).

$$F_e = \frac{1}{2\beta}\left[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}\right] \qquad (Eq.\ C4.1.2\text{-}1)$$

$$\beta = 1 - (x_o / r_o)^2 \qquad (Eq.\ C4.1.2\text{-}3)$$

$$= 1 - (-1.11/2.46)^2$$

$\beta = 0.796$

$$\sigma_{ex} = \frac{\pi^2 E}{\left(K_x L_x / r_x\right)^2} \qquad (Eq.\ C3.1.2.1\text{-}11)$$

$$= \frac{\pi^2 (29500)}{\left[(1.0)(96.0)/2.11\right]^2}$$

$= 140.7$ ksi

$$\sigma_t = \frac{1}{A r_o^{\,2}}\left[GJ + \frac{\pi^2 E C_w}{(K_t L_t)^2}\right] \qquad (Eq.\ C3.1.2.1\text{-}9)$$

$$= \frac{1}{(0.327)(2.46)^2}\left[(11300)(0.000130) + \frac{\pi^2 (29500)(0.713)}{\left[(1.0)(48.0)\right]^2}\right]$$

$= 46.27$ ksi

$$F_e = \frac{1}{(2)(0.796)}\left[(140.7 + 46.27) - \sqrt{(140.7 + 46.27)^2 - (4)(0.796)(140.7)(46.27)}\right] \qquad (Eq.\ C4.1.2\text{-}1)$$

$= 42.52$ ksi

c) Determine controlling buckling mode.

42.52 ksi < 43.84 ksi, therefore flexural–torsional buckling governs

$F_e = 42.52$ ksi

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C4.1\text{-}4)$$

$$= \sqrt{\frac{33}{42.52}}$$

$= 0.851 < 1.5$; therefore,

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \qquad (Eq.\ C4.1\text{-}2)$$

$$= \left[0.658^{(0.851)^2}\right]33$$

$$= 24.37 \text{ ksi}$$

d) Compute effective area at $f = F_n = 24.37$ ksi

Check flange as a uniformly compressed element with an edge stiffener.

$$w = 1.625 - 2(0.0765 + 0.0346) = 1.403 \text{ in.}$$

$$w/t = 1.403/0.0346 = 40.5$$

$$S = 1.28\sqrt{E/f} \qquad (Eq.\ B4\text{-}7)$$

$$= 1.28\sqrt{29500/24.37} = 44.5 \text{; therefore, } w/t \geq 0.328S \Rightarrow \text{check effective width of flange}$$

Compute flange k based on stiffener lip properties.

$$I_a = 399t^4\left[\frac{w/t}{S} - 0.328\right]^3 \leq t^4\left[115\frac{w/t}{S} + 5\right] \qquad (Eq.\ B4\text{-}8)$$

$$= 399(0.0346)^4\left[\frac{40.5}{44.5} - 0.328\right]^3 \leq (0.0346)^4\left[115\left(\frac{40.5}{44.5}\right) + 5\right]$$

$$= 0.000128 \text{ in.}^4 < 0.000149 \text{ in.}^4 \text{; therefore, } I_a = 0.000128 \text{ in.}^4$$

$$d = 0.500 - 0.0765 - 0.0346 = 0.389 \text{ in.}$$

$$\theta = 90 \text{ degrees}$$

$$I_s = \left(d^3 t \sin^2\theta\right)/12 \qquad (Eq.\ B4\text{-}10)$$

$$= (0.389)^3(0.0346)\sin^2(90°)/12 = 0.000170 \text{ in.}^4$$

$$R_I = I_s/I_a \leq 1 \qquad (Eq.\ B4\text{-}9)$$

$$= 0.000170/0.000128 = 1.33 > 1 \text{; therefore, } R_I = 1.0$$

$$D/w = 0.500/1.403 = 0.356 \qquad \text{(From Table B4-1)}$$

$0.25 < D/w \leq 0.8$; therefore,

$$k = \left(4.82 - \frac{5D}{w}\right)(R_I)^n + 0.43 \leq 4 \qquad \text{(From Table B4-1)}$$

$$= \left(4.82 - \frac{(5)(0.500)}{1.403}\right)(1.0)^n + 0.43 = 3.47 < 4 \text{ OK}$$

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu)^2}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 3.47\frac{\pi^2(29500)}{12(1-0.3)^2}\left(\frac{1}{40.5}\right)^2 = 56.41 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{24.37}{56.41}} = 0.623 < 0.673 \text{; therefore, the flange is fully effective.}$$

Check stiffener lip using Section B3.1

$f = 24.37$ ksi

$k = 0.43$

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu)^2}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 0.43\frac{\pi^2(29500)}{12(1-0.3)^2}\left(\frac{0.0346}{0.389}\right)^2 = 90.70 \text{ ksi}$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{24.37}{90.70}} = 0.518 < 0.673 \therefore \text{ lip is fully effective}$$

Check web with punchout

Per Section B2.2, treat web as two unstiffened elements, one on each side of the 1.50 inch wide punchout.

$$w = \left[5.50 - 2(0.0765 + 0.0346) - 1.50\right]/2 = 1.889 \text{ in.}$$

$k = 0.43$

$$F_{cr} = 0.43\frac{\pi^2(29500)}{12(1-0.3)^2}\left(\frac{0.0346}{1.889}\right)^2 = 3.846 \text{ ksi} \qquad (Eq.\ B2.1\text{-}5)$$

$$\lambda = \sqrt{\frac{f}{F_{cr}}} \qquad (Eq.\ B2.1\text{-}4)$$

$$= \sqrt{\frac{24.37}{3.846}} = 2.517 > 0.673$$

$$\rho = (1 - 0.22/\lambda)/\lambda \qquad (Eq.\ B2.1\text{-}3)$$

$$= (1 - 0.22/2.517)/2.517 = 0.363$$

$$b = \rho w \qquad (Eq.\ B2.1\text{-}2)$$

$$b = (0.363)(1.889) = 0.686 \text{ in.}$$

Compute A_e by subtracting the hole and ineffective area of the web from the gross section.

$$A_e = 0.327 - (0.0346)\left[1.50 + (2)(1.889 - 0.686)\right]$$

$$= 0.192 \text{ in.}^2$$

e) Compute P_n

$$P_n = A_e F_n \qquad (Eq.\ C4.1\text{-}1)$$

$$= (0.192)(24.37)$$

$P_n = 4.68$ kips

Alternatively, P_n may be interpolated from Table III-8. For the case of a 550S162-33 with a yield stress of 33 ksi, unbraced lengths of 9.0 feet and 7.0 feet in the x-axis and braced at mid-span, $P_n = 4.25$ kips and 4.90 kips, respectively. Interpolating for $KL_x = 8.0$ feet,

$$P_n = 4.25 + \frac{9.0-8.0}{9.0-7.0}(4.90 - 4.25) = 4.58 \text{ kips} \simeq 4.68 \text{ kips}$$

f) Compute maximum permissible applied live load, P_{live}

ASD

$$P_{live} \le \frac{P_n}{\Omega_c} \text{ where } \Omega_c = 1.80 \qquad (Eq.\ A4.1.1\text{-}1)$$

$$\le \frac{4.68}{1.80}$$

$$P_{live} \le 2.60 \text{ kips}$$

LRFD

$$1.6P_{live} \le \phi_c P_n \text{ where } \phi_c = 0.85 \qquad (Eq.\ A5.1.1\text{-}1)$$

$$P_{live} \le \frac{(0.85)(4.68)}{1.6}$$

$$\le 2.49 \text{ kips}$$

Required strength and stiffness of midspan brace (Section D3.3-1)

a) Calculate required brace strength

The required bracing strength calculated by *Eq.* D3.3-1 is appropriate for LRFD design, but is conservative for ASD design. Where permitted by the authority having jurisdiction, the required bracing strength for ASD designs may be reduced by the factor 1/1.50, as shown below.

ASD

$$P_{br,1} = \frac{0.01P_n}{1.5}$$

$$= \frac{0.01(4.68)}{1.5} = 0.0312 \text{ kips}$$

LRFD

$$P_{br,1} = 0.01P_n \qquad (Eq.\ D3.3\text{-}1)$$

$$= 0.01(4.68) = 0.0468 \text{ kips}$$

b) Calculate required brace stiffness

$$\beta_{br,1} = \frac{2\left[4-(2/n)\right]P_n}{L_b} \qquad (Eq.\ D3.3\text{-}2)$$

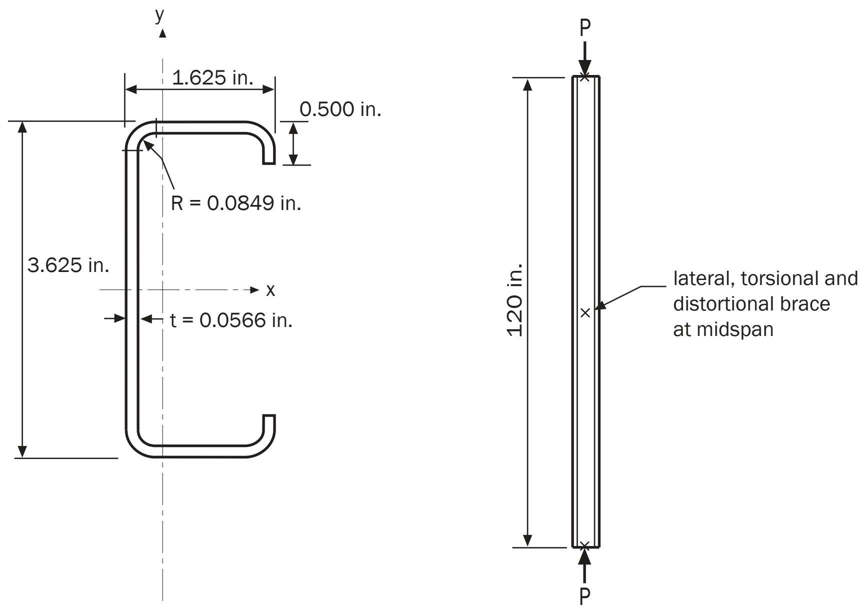
n = 1 brace location

L_b = 48.0 in.

$$\beta_{br,1} = \frac{2\left[4-(2/1)\right]4.68}{48.0} = 0.390 \text{ kips per inch} \qquad (Eq.\ D3.3\text{-}2)$$

The required stiffness is the same for both ASD and LRFD.

Example III-3: C-Section Subject to Distortional Buckling – Compression



$A = 0.422$ in.^{2}
$r_x = 1.44$ in.
$r_y = 0.605$ in.
$J = 0.000451$ in.^{4}
$C_w = 0.457$ in.^{6}
$x_o = -1.28$ in.
$r_o = 2.02$ in.

Given:

1. Steel: F_y = 50 ksi
2. Section: 362S162-54 as shown above
3. Concentrically loaded
4. Braced against buckling about the x-axis at ends only
5. Braced against buckling about the y-axis and for torsion at ends and mid-span
6. $K_x = K_y = K_t = 1.0$

Required:

1. Calculate the available strength using ASD and LRFD methods using the "all steel design" approach as described in Section D4.1 of the *Specification*. Consider distortional buckling.

Solution:

The available strength is the lower value calculated in accordance with sections C4.1 (nominal strength for yielding, flexural, flexural-torsional and torsional buckling) and C4.2 (distortional buckling strength).

1. Nominal section strength – Section C4.1

Compute the nominal axial strength using the least value of F_e for the limit states of flexural buckling (from Section C4.1.1) and flexural-torsional buckling (from Section C4.1.2).

a) Check flexural buckling (Section C4.1.1).

$$\frac{K_x L_x}{r_x} = \frac{(1.0)(120)}{1.44} = 83.3$$

$$\frac{K_y L_y}{r_y} = \frac{(1.0)(60.0)}{0.605} = 99.2$$

Since $\dfrac{K_y L_y}{r_y} > \dfrac{K_x L_x}{r_x}$, flexural buckling about the y-axis will control.

$$F_e = \frac{\pi^2 E}{\left(K_y L_y / r_y\right)^2} \qquad (Eq.\ C4.1.1\text{-}1)$$

$$= \frac{\pi^2 (29500)}{(99.2)^2} = 29.6 \text{ ksi}$$

b) Check flexural–torsional buckling (Section C4.1.2).

$$F_e = \frac{1}{2\beta}\left[\left(\sigma_{ex} + \sigma_t\right) - \sqrt{\left(\sigma_{ex} + \sigma_t\right)^2 - 4\beta\sigma_{ex}\sigma_t}\right] \qquad (Eq.\ C4.1.2\text{-}1)$$

where

$$\beta = 1 - \left(x_o / r_o\right)^2 \qquad (Eq.\ C4.1.2\text{-}3)$$

$$= 1 - (-1.28/2.02)^2$$

$$\beta = 0.598$$

$$\sigma_{ex} = \frac{\pi^2 E}{\left(K_x L_x / r_x\right)^2} \qquad (Eq.\ C3.1.2.1\text{-}11)$$

$$= \frac{\pi^2 (29500)}{(83.3)^2}$$

$$= 42.0 \text{ ksi}$$

$$\sigma_t = \frac{1}{A r_o^2}\left[GJ + \frac{\pi^2 E C_w}{\left(K_t L_t\right)^2}\right] \qquad (Eq.\ C3.1.2.1\text{-}9)$$

$$= \frac{1}{(0.422)(2.02)^2}\left[(11300)(0.000451) + \frac{\pi^2 (29500)(0.457)}{\left[(1.0)(60.0)\right]^2}\right]$$

$$= 24.4 \text{ ksi}$$

$$F_e = \frac{1}{(2)(0.598)}\left[(42.0 + 24.4) - \sqrt{(42.0 + 24.4)^2 - (4)(0.598)(42.0)(24.4)}\right] \qquad (Eq.\ C4.1.2\text{-}1)$$

$$= 18.5 \text{ ksi}$$

c) Determine controlling buckling mode.

18.5 ksi < 29.6 ksi; therefore, flexural–torsional buckling governs.

F_e = 18.5 ksi

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C4.1\text{-}4)$$

$$= \sqrt{\frac{50}{18.5}}$$

= 1.64 > 1.5; therefore,

$$F_n = \left[\frac{0.877}{\lambda_c^{\ 2}}\right]F_y \qquad (Eq.\ C4.1\text{-}3)$$

$$= \left[\frac{0.877}{(1.64)^2}\right]50$$

$$= 16.3\ \text{ksi}$$

d) Calculate the effective area at $f = F_n = 16.3$ ksi.

It can be established by calculations not shown that at $f = F_n = 16.3$ ksi, the section is fully effective; therefore, $A_e = A_g = 0.422\ \text{in.}^2$.

e) Nominal axial strength, P_n

$$P_n = A_eF_n = 0.422(16.3) = 6.88\ \text{kips} \qquad (Eq.\ C4.1\text{-}1)$$

f) Available strengths

ASD allowable strength

$$\frac{P_n}{\Omega_c} = \frac{6.88}{1.80} = 3.82\ \text{kips}$$

LRFD design strength

$$\phi_cP_n = 0.85(6.88) = 5.85\ \text{kips}$$

2. Distortional buckling strength – Section C4.2

The available distortional buckling strength is calculated using Section C4.2. The strength is a function of the elastic critical distortional buckling load, P_{crd}, and the load at first yield, P_y, as defined in Equations C4.2-1 through C4.2-5. The bulk of Section C4.2 (subsections a, b and c) is devoted to the determination of F_d, the elastic critical distortional buckling stress.

Calculate and compare the distortional buckling capacity predicted by all 3 subsections of C4.2, i.e. C4.2(a), C4.2(b) and C4.2(c).

Distortional buckling using Section C4.2(a)

C4.2(a) provides a conservative, simplified method for determination of F_d which can quickly determine situations when distortional buckling will not control the available strength. The 362S162-54 section meets the dimensional limits of C4.2(a)

$$F_d = \alpha k_d \frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{b_o}\right)^2 \qquad (Eq.\ C4.2\text{-}6)$$

where α accounts for discrete restraints that restrict distortional buckling at a length less than the distortional buckling half-wave, L_{cr}.

a) Calculate L_{cr} to determine whether the bracing provided at midspan provides distortional buckling restraint.

$$L_{cr} = 1.2h_o\left(\frac{b_o D\sin\theta}{h_o t}\right)^{0.6} \le 10h_o \qquad (Eq.\ C4.2\text{-}8)$$

$$= 1.2(3.625)\left(\frac{1.625(0.500)(1.0)}{3.625(0.0566)}\right)^{0.6} \le 10(3.625)$$

$$= 9.93 \text{ in.} < 36.3 \text{ in.}$$

Since the bracing spacing of 60.0 in. exceeds 9.93 in., α = 1.0 and the bracing provides no distortional buckling benefit. If continuous bracing was provided, the use of Section C4.2(b) or (c) is recommended to take advantage of the bracing.

b) Calculate k_d, the plate buckling coefficient

$$k_d = 0.05 \le 0.1\left(\frac{b_o D\sin\theta}{h_o t}\right)^{1.4} \le 8.0 \qquad (Eq.\ C4.2\text{-}9)$$

$$= 0.05 \le 0.1\left(\frac{1.625(0.500)(1.0)}{3.625(0.0566)}\right)^{1.4} \le 8.0$$

$= 0.05 \le 0.687 \le 8.0$; therefore, use k_d = 0.687

c) Calculate F_d, the elastic distortional buckling stress

$$F_d = \alpha k_d \frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{b_o}\right)^2 \qquad (Eq.\ C4.2\text{-}6)$$

$$= 1.0(0.687)\frac{\pi^2 29500}{12\left(1-(0.3)^2\right)}\left(\frac{0.0566}{1.625}\right)^2 = 22.2 \text{ ksi}$$

d) Calculate P_n, the nominal distortional buckling strength

$$P_y = A_g F_y \qquad (Eq.\ C4.2\text{-}4)$$

$$= (0.422)(50) = 21.1 \text{ kips}$$

$$P_{crd} = A_g F_d \qquad (Eq.\ C4.2\text{-}5)$$

$$= (0.422)(22.2) = 9.37 \text{ kips}$$

$$\lambda_d = \sqrt{\frac{P_y}{P_{crd}}} \qquad (Eq.\ C4.2\text{-}3)$$

$$= \sqrt{\frac{21.1}{9.37}} = 1.50$$

Since $\lambda_d > 0.561$

$$P_n = \left(1-0.25\left(\frac{P_{crd}}{P_y}\right)^{0.6}\right)\left(\frac{P_{crd}}{P_y}\right)^{0.6} P_y \qquad (Eq.\ C4.2\text{-}2)$$

$$= \left(1-0.25\left(\frac{9.37}{21.1}\right)^{0.6}\right)\left(\frac{9.37}{21.1}\right)^{0.6} 21.1 = 11.0 \text{ kips}$$

e) Available strengths for distortional buckling

ASD allowable strength

$\frac{P_n}{\Omega_c} = \frac{11.0}{1.80} = 6.11\,\text{kips}$ > 3.82 kips ; therefore, flexural-torsional buckling controls.

LRFD design strength

$\phi_c P_n = 0.85(11.0) = 9.35\,\text{kips}$ > 5.85 kips; therefore flexural-torsional buckling controls

The available strengths for the distortional buckling limit state exceed those for flexural-torsional buckling; therefore, a more refined distortional buckling analysis is unnecessary. The remainder of the example demonstrates the more refined calculations that could be done if distortional buckling controlled and a less conservative result was desired.

Distortional buckling using Section C4.2(b)

Section C4.2(b) provides a more precise, but more involved calculation for the elastic distortional buckling stress, F_d. Tabulated geometric properties are provided in Tables III-4 through III-6 for standard sections; however, the method is still quite involved and engineers are instead encouraged to use Section C4.2(c) with a computational analysis instead of this hand method.

The analytical model for predicting distortional buckling in C4.2(b) considers flexural-torsional buckling of the flange as a column restrained at the web/flange juncture by the available rotational stiffness from bending/buckling of the web plate. Cross-section properties of the flange itself must be calculated. The *Specification* Commentary provides formulae for these properties in Table C-C3.1.4(b)-1. These formulae may also be found in Section 3.4 of this *Manual*. Example II-4 illustrates the calculation of the required flange section properties of a similar section in detail. In the interest of brevity, the following calculations use precomputed distortional buckling coefficients taken from Table III-5.

Determine the distortional buckling strength from Table III-5, without sheathing

If the distortional buckling unbraced length, L_m, equals or exceeds the distortional buckling half-wavelength, L_{cr}, the nominal distortional buckling strength, P_n, may be taken directly from Table III-5.

L_{cr} = 13.3 in. < 60.0 in.; therefore, (from Table III-5)

P_n = 16.7 kips (from Table III-5)

This value exceeds the distortional buckling strength calculated using Section C4.2(a) by approximately 50%. In this case, flexural-torsional buckling controls, so there is no benefit to the more refined analysis.

Determine the distortional buckling strength from Table III-5 considering sheathing.

If the member is sheathed and the rotational stiffness of the sheathing is known, additional strength can be calculated using *Eq*. C4.2-10. Assuming the member is sheathed with 7/16 in. OSB attached to both flanges with #8 fasteners at 12 in. on center, and the members are spaced at 24 in. on center, the rotational stiffness contributed by the sheathing, k_ϕ, is 0.0957 kip-in./rad/in. (from Commentary Section C3.1.4)

a) Calculate F_d, the elastic distortional buckling stress

$$F_d = \frac{k_{\phi fe} + k_{\phi we} + k_\phi}{\tilde{k}_{\phi fg} + \tilde{k}_{\phi wg}} \qquad (Eq.\ C4.2\text{-}10)$$

where

$k_{\phi fe}$ = 0.348 kip-in./rad/in. (from Table III-5)

$k_{\phi we}$ = 0.270 kip-in./rad/in. (from Table III-5)

$\tilde{k}_{\phi fg}$ = 0.00823 in.^{2} (from Table III-5)

$\tilde{k}_{\phi wg}$ = 0.00251 in.^{2} (from Table III-5)

$$F_d = \frac{0.348 + 0.270 + 0.0957}{0.00823 + 0.00251} = 66.5 \text{ ksi}$$

b) Calculate P_n, the nominal distortional buckling strength

$$P_{crd} = A_g F_d \quad (Eq.\ C4.2\text{-}5)$$

$$= (0.422)(66.5) = 28.1 \text{ kips}$$

$$\lambda_d = \sqrt{\frac{P_y}{P_{crd}}} \quad (Eq.\ C4.2\text{-}3)$$

$$= \sqrt{\frac{21.1}{28.1}} = 0.867$$

Since $\lambda_d > 0.561$

$$P_n = \left(1 - 0.25\left(\frac{P_{crd}}{P_y}\right)^{0.6}\right)\left(\frac{P_{crd}}{P_y}\right)^{0.6} P_y \quad (Eq.\ C4.2\text{-}2)$$

$$= \left(1 - 0.25\left(\frac{28.1}{21.1}\right)^{0.6}\right)\left(\frac{28.1}{21.1}\right)^{0.6} 21.1 = 17.6 \text{ kips}$$

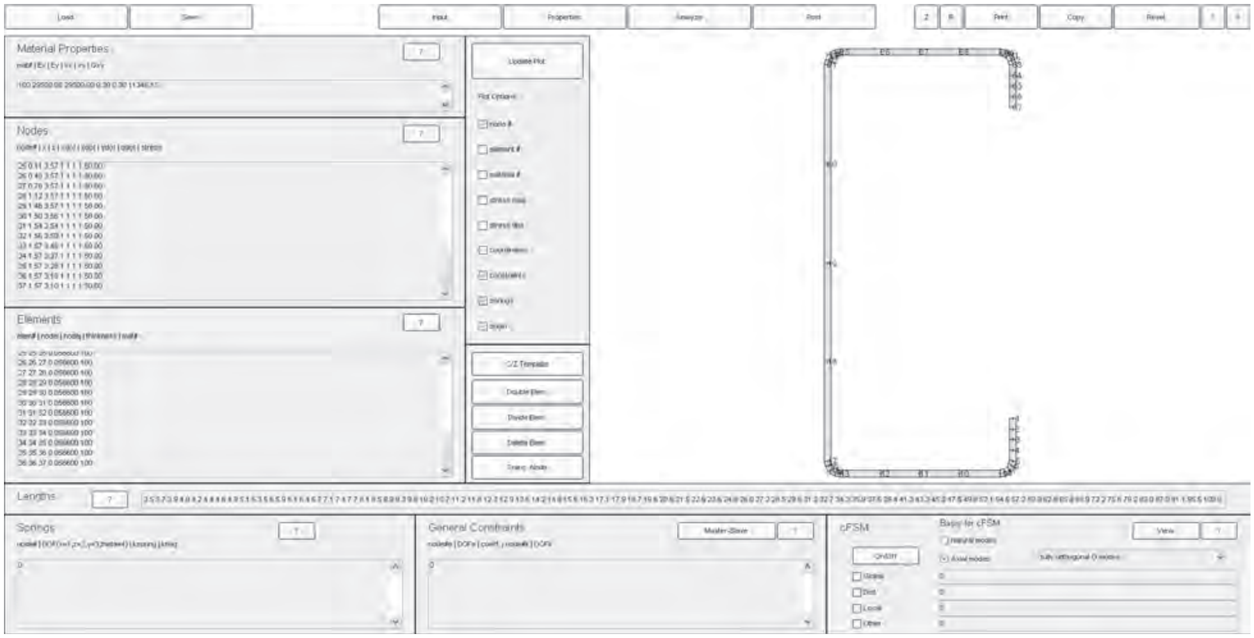
Compared with the Section C4.2(b) solution without sheathing, the nominal distortional buckling strength is increased by approximately 5% by the sheathing, but flexural-torsional buckling still controls the strength of the member.

Distortional buckling using Section C4.2(c)

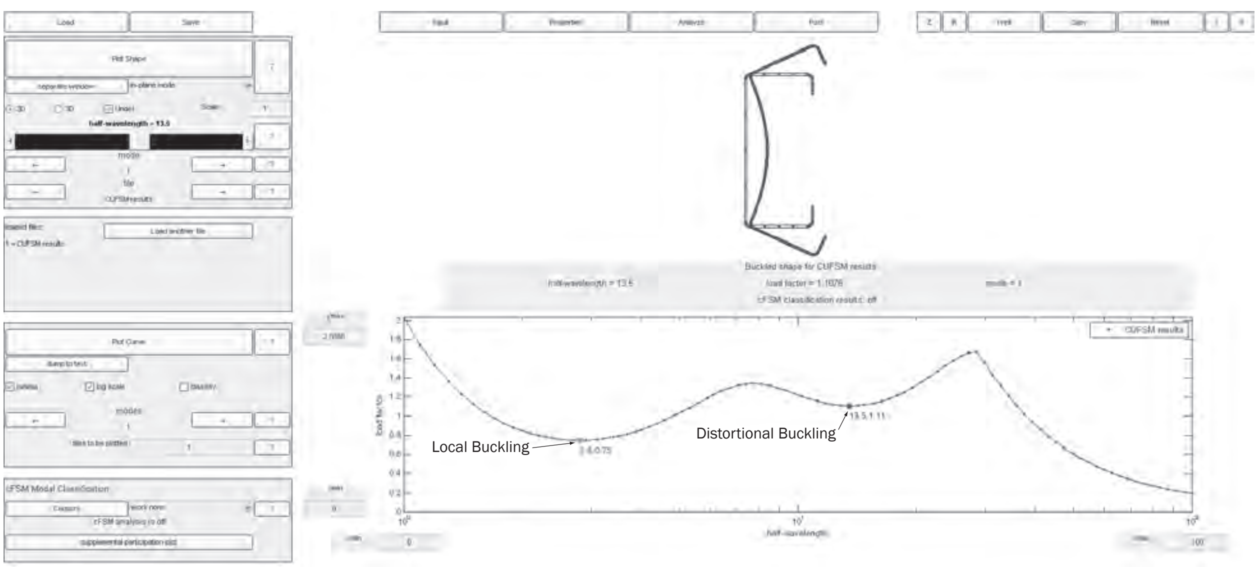
Section C4.2(c) permits the use of "rational analysis" to calculate the distortional buckling stress, F_d. The following solution uses the software program CUFSM[1]. Similar solutions can be obtained from other commercial programs.

The dimensions in the model shown below match the flat lengths and radii of the standard 362S162-54 section; consequently, the resulting gross section properties closely match those given in Table I-2. For the initial model, no rotational restraint from sheathing is included.

[1] CUFSM is a free, open source program using the semi-analytical finite strip method for determination of thin-walled member stability. The program, along with tutorials, etc., may be found at www.ce.jhu.edu/bschafer/cufsm.



The analysis is conducted with a uniform compression yield stress of 50 ksi applied to the section. The resulting buckling curve and buckling mode shape at the distortional buckling half-wavelength are shown in the figure below. Distortional buckling is found to occur at a half-wavelength of 13.5 in. and at a load factor of 1.11.



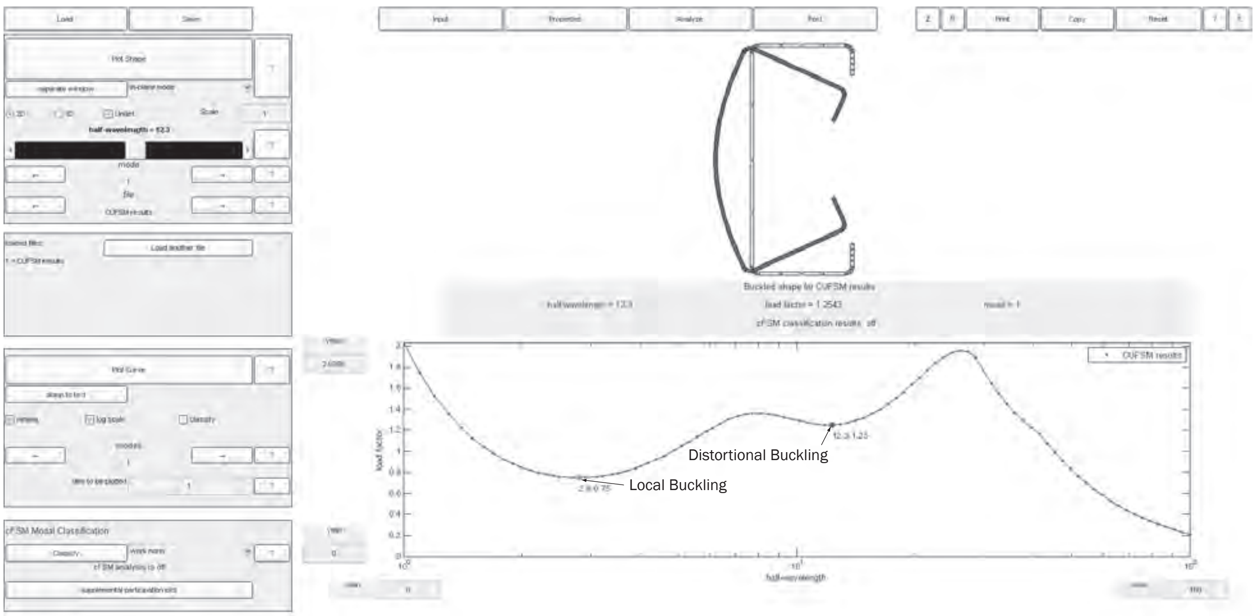
Thus, the elastic distortional buckling load, P_{crd}, is $1.11P_y$. The elastic distortional buckling stress, F_d, is calculated as:

$$F_d = \frac{P_{crd}}{A_g} = \frac{1.11P_y}{A_g}$$

$$= \frac{1.11(21.1)}{0.422} = 55.5\,\text{ksi}$$

The nominal distortional buckling strength can then be calculated from F_d using *Eqs*. C4.2-1 through C4.2-5.

If a rotational restraint of 0.057 kip-in./rad/in. is added to the model at the mid-point of the flanges, the half-wavelength decreases to 12.3 in. and the load factor increases to 1.25, as shown in the figure below.



Thus, the elastic distortional buckling load., P_{crd}, is $1.25P_y$. The elastic distortional buckling stress, F_d, is calculated as:

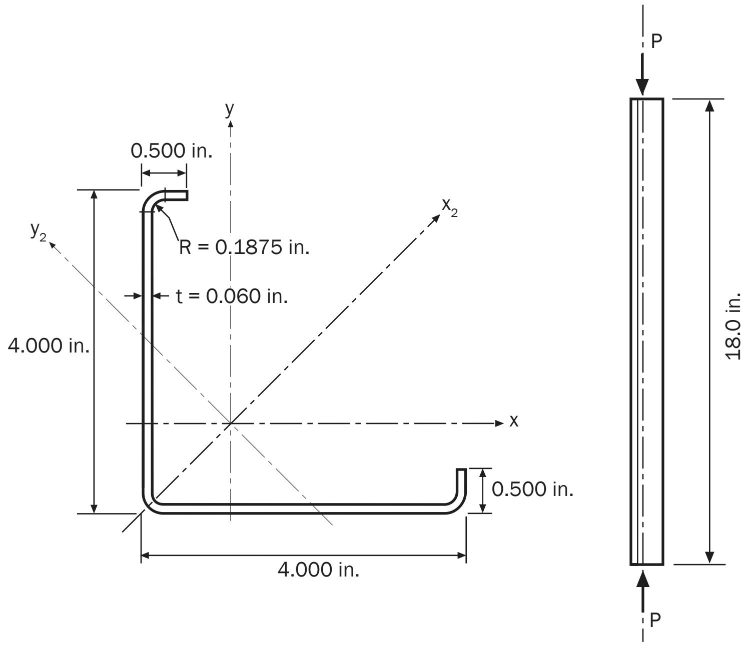
$$F_d = \frac{P_{crd}}{A_g} = \frac{1.25P_y}{A_g}$$

$$= \frac{1.25(21.1)}{0.422} = 62.5\,\text{ksi}$$

The nominal distortional buckling strength can then be calculated from F_d using *Eqs*. C4.2-1 through C4.2-5.

It can be seen that the results from the numerical analysis agree well, but not exactly, with the provisions of Section C4.2(b).

Example III-4: Unbraced Equal Leg Angle With Lips – Compression



From Table I-5 or Example I-4

$A = 0.512$ in.^{2}
$I_x = 0.960$ in.^{4}
$I_{y2} = 0.397$ in.^{4}
$r_{y2} = 0.881$ in.
$x_o = -1.633$ in.
$r_o = 2.53$ in.
$J = 0.000615$ in.^{4}
$C_w = 0.0533$ in.^{6}
$j = 3.13$ in.

Given:

1. Steel: F_y = 50 ksi
2. Section: 4LS4x060 as shown above
3. Section is concentrically loaded in compression
4. $KL_x = KL_y = KL_t = 18.0$ in.

Required:

1. ASD allowable design strength under concentric compression loading

Solution:

Nominal Axial Strength, P_n (Section C4)

The equal leg angle is a singly-symmetric section; therefore, check flexural and torsional-flexural buckling. Single angles do not exhibit distortional buckling.

a) Flexural buckling (Section C4.1.1)

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \qquad (Eq.\ C4.1.1\text{-}1)$$

use least radius of gyration, r_{y2}

$$F_e = \frac{\pi^2(29500)}{\left[(1.0)(18.0)/0.881\right]^2} = 697.5 \text{ ksi} \qquad (Eq.\ C4.1.1\text{-}1)$$

b) Torsional-flexural buckling (Section C4.1.2)

$$F_e = \frac{1}{2\beta}\left[(\sigma_{ex2}+\sigma_t)-\sqrt{(\sigma_{ex2}+\sigma_t)^2-4\beta\sigma_{ex2}\sigma_t}\right] \qquad (\text{from } Eq.\ C4.1.2\text{-}1)$$

where the x_2 axis is the axis of symmetry.

$$\beta = 1-(x_o/r_o)^2 \qquad (Eq.\ C4.1.2\text{-}3)$$

$$= 1-\left(\frac{-1.633}{2.53}\right)^2 = 0.583$$

$$\sigma_{ex2} = \frac{\pi^2 E}{(K_{x2}L_{x2}/r_{x2})^2} \qquad (\text{from } Eq.\ C3.1.2.1\text{-}11)$$

For the case of an equal leg angle, the radius of gyration about the axis of symmetry, r_{x2}, can be computed as:

$$I_{x2} = 2I_x - I_{y2}$$

$$= (2)(0.960) - 0.397 = 1.523 \text{ in.}^4$$

$$r_{x2} = \sqrt{\frac{I_{x2}}{A}}$$

$$= \sqrt{\frac{1.523}{0.512}} = 1.725 \text{ in.}$$

$$K_{x2} = K = 1.0$$

$$\sigma_{ex2} = \frac{\pi^2(29500)}{[(1.0)(18.0)/1.725]^2} \qquad (Eq.\ C3.1.2.1\text{-}11)$$

$$= 2674 \text{ ksi}$$

$$\sigma_t = \frac{1}{Ar_o^2}\left[GJ + \frac{\pi^2 EC_w}{(K_tL_t)^2}\right] \qquad (Eq.\ C3.1.2.1\text{-}9)$$

$$= \frac{1}{(0.512)(2.53)^2}\left[(11300)(0.000615) + \frac{\pi^2(29500)(0.0533)}{[(1.0)(18.0)]^2}\right]$$

$$= 16.7 \text{ ksi}$$

$$F_e = \frac{1}{(2)(0.583)}\left[(2674+16.7) - \sqrt{(2674+16.7)^2 - (4)(0.583)(2674)(16.7)}\right] \qquad (Eq.\ C4.1.2\text{-}1)$$

$$= 16.7 \text{ ksi} \qquad \text{CONTROLS}$$

c) Nominal axial strength (Section C4)

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C4.1\text{-}4)$$

$$= \sqrt{\frac{50}{16.7}} = 1.73 > 1.5$$

$$F_n = \left[\frac{0.877}{\lambda_c^2}\right]F_y \qquad (Eq.\ C4.1\text{-}3)$$

$$= \left[\frac{0.877}{(1.73)^2}\right]50 = 14.7 \text{ ksi}$$

$$P_n = A_eF_n \qquad (Eq.\ C4.1\text{-}1)$$

From Example I-11 at a uniform compression stress of 14.7 ksi,

A_e = 0.383 in.^{2}

$P_n = (0.383)(14.7) = 5.63$ kips (*Eq*. C4.1-1)

ASD Allowable Strength not considering minimum eccentricity

$\Omega_c = 1.80$

$$\frac{P_n}{\Omega_c} = \frac{5.63}{1.80} = 3.13 \text{ kips}$$

Nominal Flexural Strength, M_n (Section C3.1.2.1)

Sections C4.1(b) and C5.2.1 of the *Specification* require consideration of an eccentricity of PL/1000 about the minor axis.

The equal leg angle is a singly-symmetric section, therefore check lateral-torsional buckling about the minor principal axis, the axis perpendicular to the axis of symmetry.

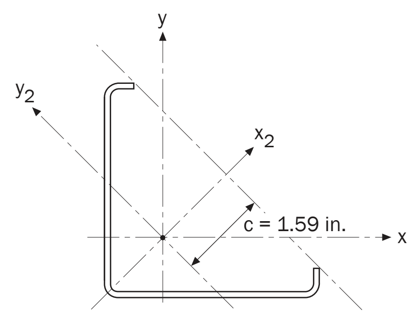
$$F_e = \frac{C_s A \sigma_{ex2}}{C_{TF} S_f}\left[j + C_s\sqrt{j^2 + r_o{}^2(\sigma_t/\sigma_{ex2})}\right] \quad \text{(from } Eq. \text{ C3.1.2.1-10)}$$

$$S_{y2} = \frac{I_{y2}}{c}$$

$$= \frac{0.397}{1.59} = 0.250 \text{ in.}^3$$

σ_t = 16.7 ksi (computed above)

σ_{ex2} = 2674 ksi (computed above)



$C_{TF} = 1$

$C_s = -1$ (assume worst case - tension on shear center side of centroid)

$$F_e = \frac{(-1)(0.512)(2674)}{(1)(0.250)}\left[3.13 + (-1)\sqrt{(3.13)^2 + (2.53)^2(16.7/2674)}\right] \quad (Eq. \text{ C3.1.2.1-10})$$

= 34.94 ksi

Since $2.78F_y > F_e > 0.56F_y$,

$$F_c = \frac{10}{9}F_y\left(1 - \frac{10F_y}{36F_e}\right) \quad (Eq. \text{ C3.1.2.1-2})$$

$$= \frac{10}{9}(50.0)\left(1 - \frac{(10)(50.0)}{(36)(34.94)}\right) = 33.47 \text{ ksi}$$

By calculations not shown, the section is found to be fully effective for bending about the y_2 axis with a maximum compression stress of 33.47 ksi at the ends of the lips; therefore,

$S_c = S_{y2}$

$M_n = S_c F_c$ (*Eq*. C3.1.2.1-1)

$= (0.250)(33.47) = 8.37$ kip-in.

ASD Allowable Strength including minimum eccentricity

M_{y2} = PL/1000

$= P(18.0)/1000 = 0.0180P$

Assume $\frac{\Omega_c P}{P_n}$ > 0.15, and solve for the allowable load, P, such that equations C5.2.1-1 and C5.2.1-2 are satisfied.

C_{my2} = 1.0

K_{y2} = K = 1.0

$$P_{Ey2} = \frac{\pi^2 EI_{y2}}{\left(K_{y2}L_{y2}\right)^2} \qquad (Eq.\ C5.2.1\text{-}7)$$

$$= \frac{\pi^2(29500)(0.397)}{\left[(1.0)(18.0)\right]^2} = 357 \text{ kips}$$

$$\alpha_{y2} = 1 - \frac{\Omega_c P}{P_{Ey2}} > 0 \qquad (Eq.\ C5.2.1\text{-}5)$$

$$= 1 - \frac{(1.80)(P)}{357}$$

$$= 1 - 0.00504P$$

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{my2} M_{y2}}{M_{ny2}\alpha_{y2}} \le 1.0 \qquad (Eq.\ C5.2.1\text{-}1)$$

$$\frac{(1.80)(P)}{5.63} + \frac{(1.67)(1.0)(0.0180P)}{(8.37)(1-0.00504P)} \le 1.0$$

Solving for P:

$P \le 3.09$ kips

By calculations not shown, similar to those in Example I-11, P_{no} is calculated at f = F_y.

P_{no} = 10.5 kips

$$\frac{\Omega_c P}{P_{no}} + \frac{\Omega_b M_{y2}}{M_{ny2}} \le 1.0 \qquad (Eq.\ C5.2.1\text{-}2)$$

$$\frac{(1.80)(P)}{10.5} + \frac{(1.67)(0.0180P)}{(8.37)} \le 1.0$$

Solving for P:

$P \le 5.71$ kips

Therefore, equation C5.2.1-1 controls

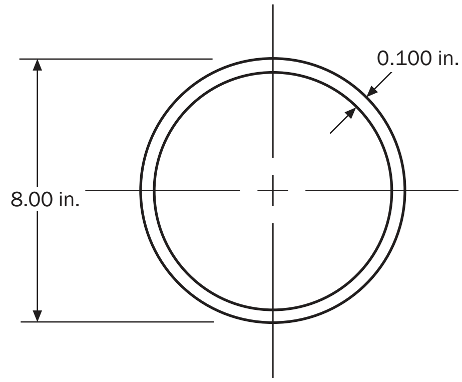
P $\le$ 3.09 kips

Check assumption that $\frac{\Omega_c P}{P_n}$ > 0.15

$$\frac{\Omega_c P}{P_n} = \frac{(1.80)(3.09)}{5.63} = 0.99 > 0.15 \quad \text{OK}$$

Therefore, the ASD allowable design strength is 3.09 kips.

Example III-5: Tubular Section - Round - Bending and Compression



Given:

1. Steel: F_y = 42 ksi
2. Section: Shown in sketch above
3. Height: L = 10.0 feet, simply supported at each end
4. Axial Loads: Dead Load: P_D = 7.5 kips, Roof Live Load: P_{Lr} = 20 kips
5. Transverse Concentrated Wind Load (at midspan): P_W = 3.0 kips

Required:

Check the adequacy of the section using ASD and LRFD methods with ASCE/SEI 7-05 load combinations.

Solution:

Nominal Axial Strength, P_n (Section C4.1.5)

Ratio of outside diameter to wall thickness

$$D/t = 8.00/0.100 = 80.0$$

$$D/t < 0.441E/F_y = 0.441(29500/42) = 310 \quad \text{OK}$$

a) Compute nominal axial stress, F_n

$$I = \frac{\pi}{4}\left[(\text{Outside Radius})^4 - (\text{Inside Radius})^4\right]$$

$$= \frac{\pi}{4}\left[(4.00)^4 - (3.90)^4\right]$$

$$= 19.37 \text{ in.}^4$$

$$A = \frac{\pi}{4}\left[(\text{Outside Diameter})^2 - (\text{Inside Diameter})^2\right]$$

$$= \frac{\pi}{4}\left[(8.00)^2 - (7.80)^2\right]$$

$$= 2.482 \text{ in.}^2$$

$$r = \sqrt{I/A}$$

$$= \sqrt{19.37/2.482} = 2.794 \text{ in.}$$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \qquad (Eq.\ C4.1.1\text{-}1)$$

$$= \frac{\pi^2(29500)}{\left[(10)(12)/2.794\right]^2} = 158 \text{ ksi}$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} = \sqrt{\frac{42}{158}} = 0.516 \qquad (Eq.\ C4.1\text{-}4)$$

Since $\lambda_c \le 1.5$

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \qquad (Eq.\ C4.1\text{-}2)$$

$$= \left(0.658^{(0.516)^2}\right)42 = 37.6 \text{ ksi}$$

b) Compute effective area, A_e

$$A_o = \left[\frac{0.037}{(DF_y)/(tE)} + 0.667\right]A \le A \qquad (Eq.\ C4.1.5\text{-}2)$$

$$= \left[\frac{0.037}{(8.000)(42)/\left[(0.100)(29500)\right]} + 0.667\right]2.482$$

$= 2.462$ in.^{2} < A

$$R = \frac{F_y}{2F_e} \le 1.0 \qquad (Eq.\ C4.1.5\text{-}3)$$

$$= \frac{42}{(2)(158)} = 0.133 < 1.0$$

$$A_e = A_o + R(A - A_o) \qquad (Eq.\ C4.1.5\text{-}1)$$

$$A_e = 2.462 + 0.133(2.482 - 2.462) = 2.465 \text{ in.}^2$$

c) Compute nominal axial strengths, P_n and P_{no}

$$P_n = F_n A_e \qquad (Eq.\ C4.1\text{-}1)$$

$$= (37.6)(2.465)$$

$= 92.7$ kips

Compute P_{no} for use in Section C5.2

$F_n = F_y$

$$P_{no} = F_y A_o \qquad (Eq.\ C4.1\text{-}1)$$

$$P_{no} = (42)(2.462) = 103 \text{ kips}$$

Nominal Flexural Strength, M_n (from Example II-7)

$M_n = 233$ kip-in.

Combined Bending and Compression

$$M_W = PL/4 = (3.0)(10)(12)/4 = 90.0 \text{ kip-in.}$$

ASD

ASCE/SEI 7-05 load combinations considered:

D + L

D + W

D + 0.75W + 0.75L_r

Controlling load combination (by inspection) is D + 0.75W + 0.75L_r

$$P = P_D + 0.75P_{Lr} = 7.5 + (0.75)(20) = 22.5 \text{ kips}$$

$$M_x = 0.75M_W = (0.75)(90.0) = 67.5 \text{ kip-in.}$$

$$\Omega_c = 1.80$$

$$\Omega_b = 1.67$$

$$\frac{\Omega_c P}{P_n} = \frac{(1.80)(22.5)}{92.7} = 0.437 > 0.15$$; therefore, use Equations C5.2.1-1 and C5.2.1-2

$$C_{mx} = 1.0$$

$$P_{Ex} = \frac{\pi^2 EI_x}{(K_x L_x)^2} \quad (Eq.\ C5.2.1\text{-}6)$$

$$= \frac{\pi^2(29500)(19.37)}{[(1.0)(120)]^2} = 392 \text{ kips}$$

$$\alpha_x = 1 - \frac{\Omega_c P}{P_{Ex}} > 0 \quad (Eq.\ C5.2.1\text{-}4)$$

$$= 1 - \frac{(1.80)(22.5)}{392} = 0.897$$

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{mx} M_x}{M_{nx}\alpha_x} \le 1.0 \quad (Eq.\ C5.2.1\text{-}1)$$

$$\frac{(1.80)(22.5)}{92.7} + \frac{(1.67)(1.0)(67.5)}{(233)(0.897)} = 0.976 < 1.0 \quad \text{OK}$$

$$\frac{\Omega_c P}{P_{no}} + \frac{\Omega_b M_x}{M_{nx}} \le 1.0 \quad (Eq.\ C5.2.1.\text{-}2)$$

$$\frac{(1.80)(22.5)}{103} + \frac{(1.67)(67.5)}{(233)} = 0.877 < 1.0 \quad \text{OK}$$

LRFD

ASCE/SEI 7-05 load combinations considered:

1.4D

1.2D + 1.6L

1.2D + 1.6W + 0.5L_r

0.9D + 1.6W

Controlling load combination (by inspection) is 1.2D + 1.6W + 0.5L_r

$$\overline{P} = P_u = 1.2P_D + 0.5P_{Lr} = (1.2)(7.5) + (0.5)(20) = 19.0 \text{ kips}$$

$$\overline{M} = M_u = 1.6M_W = (1.6)(90.0) = 144 \text{ kip-in.}$$

ϕ_c = 0.85

ϕ_b = 0.95

$\frac{\overline{P}}{\phi_c P_n} = \frac{19.0}{(0.85)(92.7)} = 0.241 > 0.15$; therefore, use Equations C5.2.2-1 and C5.2.2-2

C_{mx} = 1.0

$$\alpha_x = 1 - \frac{\overline{P}}{P_{Ex}} > 0 \qquad (Eq.\ C5.2.2\text{-}4)$$

$$= 1 - \frac{19.0}{392} = 0.952$$

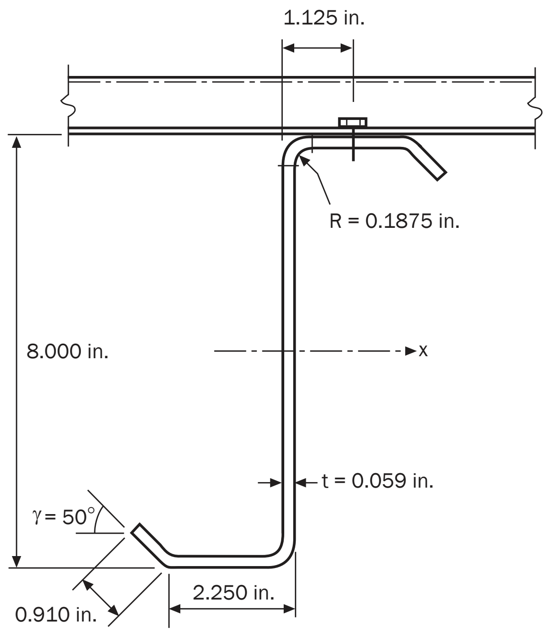
$$\frac{\overline{P}}{\phi_c P_n} + \frac{C_{mx}\overline{M}_x}{\phi_b M_{nx}\alpha_x} \le 1.0 \qquad (Eq.\ C5.2.2\text{-}1)$$

$$\frac{19.0}{(0.85)(92.7)} + \frac{(1.0)(144)}{(0.95)(233)(0.952)} = 0.924 < 1.0 \quad \text{OK}$$

$$\frac{\overline{P}}{\phi_c P_{no}} + \frac{\overline{M}_x}{\phi_b M_{nx}} \le 1.0 \qquad (Eq.\ C5.2.2\text{-}2)$$

$$\frac{19.0}{(0.85)(103)} + \frac{144}{(0.95)(233)} = 0.868 < 1.0 \quad \text{OK}$$

Example III-6: Stiffened Z-Section With One Flange Through-Fastened To Deck Or Sheathing – Compression



Given:

1. Steel: F_y = 55 ksi
2. Span = 25.0 feet = 300 in.
3. Section: 8ZS2.25x059
   d = D = 8.000 in.
   b = B = 2.250 in.
   t = 0.059 in.
   A = 0.822 $in.^2$
   r_x = 3.07 in.
4. Through-fastened at 12 in. o.c. and assumed to be located at the center of the flange.
5. Panel has a rotational stiffness of 0.002 kip/in./in. determined by tests performed in accordance with AISI S901-08[2].
6. Both flanges are restrained from lateral movement at the supports.

Required:

Available compression strengths using ASD and LRFD

Solution:

Nominal Axial Strength, P_n - Flexural Buckling about the X-axis (Section C4.1.1)

K = 1

[2] AISI S901-08, *Rotational Lateral Test Method for Beam to Panel Assemblies*, American Iron and Steel Institute, Washington, D.C., 2008

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \qquad (Eq.\ C4.1.1\text{-}1)$$

$$= \frac{\pi^2 (29500)}{(300/3.07)^2} = 30.5 \text{ ksi}$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C4.1\text{-}4)$$

$$= \sqrt{\frac{55}{30.5}} = 1.34 < 1.50$$

$$F_n = \left(0.658^{\lambda_c^2}\right) F_y \qquad (Eq.\ C4.1\text{-}2)$$

$$= \left(0.658^{1.34^2}\right) 55 = 25.9 \text{ ksi}$$

$$P_n = A_e F_n \qquad (Eq.\ C4.1\text{-}1)$$

In Example I-10, A_e was calculated as 0.578 in.^{2} at f = 25.9 ksi

$$P_n = (0.578)(25.9) = 15.0 \text{ kips}$$

Nominal Axial Strength, P_n - Flexural-Torsional Buckling (Section D6.1.3)

a) Check limits of applicability of Section D6.1.3

(1) $t \le 0.125$ in.; t = 0.059 in. OK

(2) $6\text{ in.} \le d \le 12\text{ in.}$; d = 8.00 in. OK

(3) Flanges are edge stiffened compression elements OK

(4) $70 \le d/t \le 170$; $d/t = 8.00/0.059 = 136$ OK

(5) $2.8 \le d/b \le 5$; $d/b = 8.00/2.25 = 3.56$ OK

(6) $16 \le \dfrac{\text{flat flange width}}{t} \le 50$; $\dfrac{1.889}{0.059} = 32.0$ (from Example I-3) OK

(7) Both flanges are prevented from moving laterally at the supports OK

(8) Fastener spacing ≤ 12 in. OK

Rotational lateral stiffness ≥ 0.0015 kip/in./in. OK

(9) $F_y \ge 33$ ksi OK

(10) Span length ≤ 33 ft; L = 25 ft OK

All conditions are satisfied

b) Compute P_n

$$P_n = C_1 C_2 C_3 A E/29500 \qquad (Eq.\ D6.1.3\text{-}1)$$

α = 1 (units are inches)

$$x = a/b \qquad (Eq.\ D6.1.3\text{-}5)$$

$$= 1.125/2.25 = 0.50$$

$$C_1 = 0.79x + 0.54 \qquad (Eq.\ D6.1.3\text{-}2)$$

$$= (0.79)(0.50) + 0.54 = 0.935$$

$$C_2 = 1.17\alpha t + 0.93 \qquad (Eq.\ D6.1.3\text{-}3)$$

$$= (1.17)(1)(0.059) + 0.93 = 1.00$$

$$C_3 = \alpha(2.5b - 1.63d) + 22.8 \qquad (Eq.\ D6.1.3\text{-}4)$$

$$= 1\left[(2.5)(2.25) - (1.63)(8.00)\right] + 22.8 = 15.4$$

$$P_n = (0.935)(1.00)(15.4)(0.822)(29500)/29500 \qquad (Eq.\ D6.1.3\text{-}1)$$

$$= 11.8 \text{ kips}$$

Governing Limit State is Flexural-Torsional Buckling

P_n = minimum of 11.8 kips or 15.0 kips = 11.8 kips

Available Strength

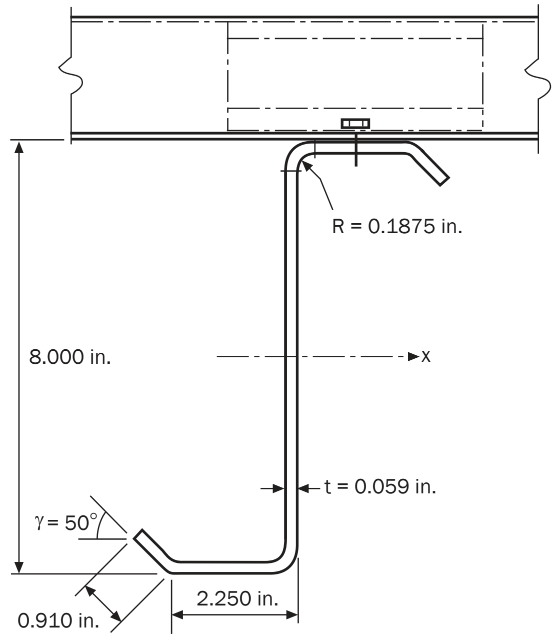
ASD

$$\frac{P_n}{\Omega} = \frac{11.8}{1.80} = 6.56 \text{ kips}$$

LRFD

$$\phi P_n = (0.85)(11.8) = 10.0 \text{ kips}$$

Example III-7: Stiffened Z-Section With One Flange Fastened To a Standing Seam Roof – Compression



Given:

1. Steel: F_y = 55 ksi
2. Span = 25.0 ft = 300 in.
3. Section: 8ZS2.25x059

   d = D = 8.000 in.

   b = B = 2.250 in.

   t = 0.059 in.

   A = 0.822 in.^{2}

   r_x = 3.07 in.
4. Reduction factor, R = 0.70, determined from uplift tests performed in accordance with AISI S908-08[3]
5. Both flanges are restrained from lateral movement at the supports.

Required:

Available compression strengths using ASD and LRFD

Solution:

Nominal Axial Strength, P_n - Flexural Buckling about the X-axis (Section C4.1.1)

K = 1.0

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \qquad (Eq.\ C4.1.1\text{-}1)$$

$$= \frac{\pi^2(29500)}{(300/3.07)^2} = 30.5 \text{ ksi}$$

[3] AISI S908-08, *Base Test Method for Purlins Supporting a Standing Seam Roof and the Commentary*, American Iron and Steel Institute, Washington, D.C., 2008

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C4.1\text{-}4)$$

$$= \sqrt{\frac{55}{30.5}} = 1.34 < 1.50$$

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \qquad (Eq.\ C4.1\text{-}2)$$

$$= \left(0.658^{1.34^2}\right)55 = 25.9 \text{ ksi}$$

$$P_n = A_eF_n \qquad (Eq.\ C4.1\text{-}1)$$

In Example I-10, A_e was calculated as 0.578 in.² at f = 25.9 ksi

$$P_n = (0.578)(25.9) = 15.0 \text{ kips}$$

Nominal Axial Strength, P_n - Flexural-Torsional Buckling (Section D6.1.4)

a) Check limits of applicability of Section D6.1.4 (in Appendix A)

(1) $0.054 \text{ in.} \le t \le 0.125 \text{ in.}$; t = 0.059 in. OK

(2) $6 \text{ in.} \le d \le 12 \text{ in.}$; d = 8.00 in. OK

(3) Flanges are edge stiffened compression elements OK

(4) $70 \le d/t \le 170$; $d/t = 8.00/0.059 = 136$ OK

(5) $2.8 \le d/b < 5$; $d/b = 8.00/2.25 = 3.56$ OK

(6) $16 \le \dfrac{\text{flat flange width}}{t} < 50$; $\dfrac{1.889}{0.059} = 32.0$ (from Example I-3) OK

(7) Both flanges are prevented from moving laterally at the supports OK

(8) $F_y \le 70 \text{ ksi}$ OK

All conditions are satisfied.

b) Compute P_n

$d/t = 136 > 130$; therefore,

$$k_{af} = 0.20$$

$$Pn = k_{af}RF_yA \qquad (Eq.\ D6.1.4\text{-}1)$$

$$= (0.20)(0.70)(55)(0.822) = 6.33 \text{ kips}$$

Governing Limit State is Flexural-Torsional Buckling

P_n = minimum of (6.33 kips or 15.0 kips) = 6.33 kips

Available Strength

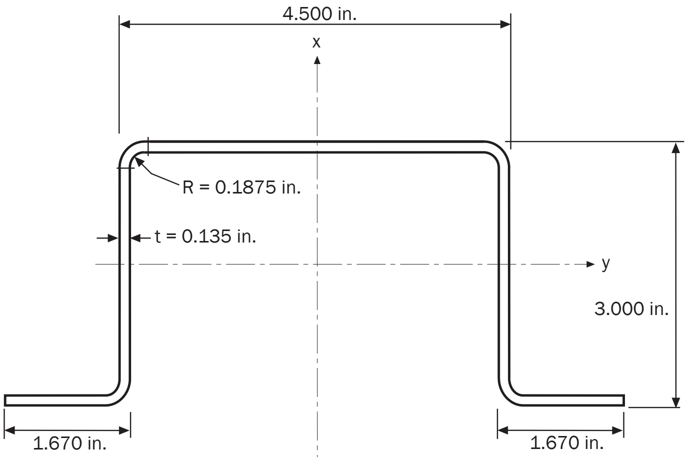
ASD

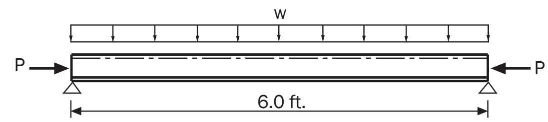
$$\frac{P_n}{\Omega} = \frac{6.33}{1.80} = 3.52 \text{ kips}$$

LRFD

$$\phi P_n = (0.85)(6.33) = 5.38 \text{ kips}$$

Example III-8: Hat Section - Bending and Compression





Given:

1. Steel: F_y = 50 ksi
2. Section: 3HU4.5x135 as shown above. From Table I-8,
   A_g = 1.74 in.^{2}
   I_y = 2.47 in.^{4}
   r_y = 1.19 in.
3. L = 6.0 feet, simply supported with continuous lateral and torsional and distortional bracing of the compression flanges.
4. Axial Loads: Dead Load: P_D = 2 kips, Live Load: P_L = 10 kips
5. Transverse Uniform Flexural Loads:
   Dead Load: w_D = 0.090 kips/ft
   Live Load: w_L = 0.360 kips/ft

Required:

Check the adequacy of the section using ASD and LRFD methods. Do not use inelastic reserve capacity.

Solution:

Bending Moments at Service Level

$$M_D = \frac{w_D L^2}{8} = \frac{(0.090)(6)^2}{8} = 0.405 \text{ kip-ft.} = 4.86 \text{ kip-in.}$$

$$M_L = \frac{w_L L^2}{8} = \frac{(0.360)(6)^2}{8} = 1.62 \text{ kip-ft.} = 19.4 \text{ kip-in.}$$

Nominal Flexural Strength, M_n

From Example II-6
M_n = 76.0 kip-in.

Nominal Axial Strength, P_n

The member is free to buckle only in the plane perpendicular to the flange.

$$F_e = \frac{\pi^2 E}{\left(KL_y/r_y\right)^2} \quad (Eq.\ C4.1.1\text{-}1)$$

$$= \frac{\pi^2(29500)}{\left[(1.0)(72.0)/1.19\right]^2} = 79.5 \text{ ksi}$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \quad (Eq.\ C4.1\text{-}4)$$

$$= \sqrt{\frac{50.0}{79.5}} = 0.793 \le 1.5$$

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \quad (Eq.\ C4.1\text{-}2)$$

$$= \left[0.658^{(0.793)^2}\right]50 = 38.4 \text{ ksi}$$

$$P_n = A_eF_n \quad (Eq.\ C4.1\text{-}1)$$

From Example I-13

A_e = 1.74 in.² at a stress level of 50 ksi.

All elements are fully effective. By inspection, all elements will therefore be fully effective at the lower stress of 38.4 ksi:

A_e = A_{gross} = 1.74 in.²

$$P_n = (1.74)(38.4) = 66.8 \text{ kips} \quad (Eq.\ C4.1\text{-}1)$$

Combined Compression and Bending - ASD (Section C5.2.1)

a) Required Axial Strength

$P = P_D + P_L = 2.0 + 10.0 = 12.0$ kips

b) Required Flexural Strength

$M_y = M_D + M_L = 4.86 + 19.4 = 24.3$ kip-in.

c) Combined Strength

Check Ω_cP/P_n

Ω_c = 1.80

$$\frac{\Omega_cP}{P_n} = \frac{(1.80)(12.0)}{66.8} = 0.323 > 0.15\text{; therefore, check Equations C5.2.1-1 and C5.2.1-2.}$$

C_{my} = 1.0

$$P_{Ey} = \frac{\pi^2 EI_y}{\left(K_yL_y\right)^2} \quad (Eq.\ C5.2.1\text{-}7)$$

$$= \frac{\pi^2(29500)(2.47)}{\left[(1.0)(72.0)\right]^2} = 138.7 \text{ kips}$$

$$\alpha_y = 1 - \frac{\Omega_cP}{P_{Ey}} > 0 \quad (Eq.\ C5.2.1\text{-}5)$$

$$\alpha_y = 1 - \frac{(1.80)(12.0)}{138.7} = 0.844 \quad (Eq.\ C5.2.1\text{-}5)$$

M_x = 0.0

Ω_b = 1.67

$$\frac{\Omega_c P}{P_n}+\frac{\Omega_b C_{mx} M_x}{M_{nx}\alpha_x}+\frac{\Omega_b C_{my} M_y}{M_{ny}\alpha_y} \le 1.0 \quad (Eq.\ C5.2.1\text{-}1)$$

$$\frac{(1.80)(12.0)}{66.8}+\frac{(1.67)(1.0)(24.3)}{(76.0)(0.844)} \le 1.0$$

$$0.323+0.633=0.956<1.0 \quad \text{OK}$$

$$\frac{\Omega_c P}{P_{no}}+\frac{\Omega_b M_x}{M_{nx}}+\frac{\Omega_b M_y}{M_{ny}} \le 1.0 \quad (Eq.\ C5.2.1\text{-}2)$$

P_{no} = A_eF_y where A_e was calculated as 1.74 in.^{2} in Example I-13 at a stress level of 50 ksi.

= $(1.74)(50)=87.0$ kips

$$\frac{(1.80)(12.0)}{87.0}+\frac{(1.67)(24.3)}{76.0} = 0.248 + 0.534 = 0.782 < 1.0 \quad \text{OK} \quad (Eq.\ C5.2.1\text{-}2)$$

Combined Compression and Bending - LRFD (Section C5.2.2)

a) Required Axial Strength

$\overline{P}$ = $P_u = 1.2P_D + 1.6P_L$

= $(1.2)(2.0)+(1.6)(10.0)=18.4$ kips

b) Required Flexural Strength

$\overline{M}_y$ = $M_{uy} = 1.2M_D + 1.6M_L$

= $(1.2)(4.86)+(1.6)(19.4)=36.9$ kip-in.

c) Combined Strength

Check $\overline{P}/\phi_c P_n$

ϕ_c = 0.85

$$\frac{\overline{P}}{\phi_c P_n} = \frac{18.4}{(0.85)(66.8)} = 0.324 > 0.15;\ \text{therefore, check Equations C5.2.2-1 and C5.2.2-2.}$$

C_{my} = 1.0

$$\alpha_y = 1-\frac{\overline{P}}{P_{Ey}}>0 \quad (Eq.\ C5.2.2\text{-}5)$$

P_{Ey} = 138.7 kips (calculated above)

$$\alpha_y = 1-\frac{18.4}{138.7} = 0.867 \quad (Eq.\ C5.2.2\text{-}5)$$

M_x = 0.0

ϕ_b = 0.95 (laterally braced beam)

$$\frac{\overline{P}}{\phi_c P_n}+\frac{C_{mx}\overline{M}_x}{\phi_b M_{nx}\alpha_x}+\frac{C_{my}\overline{M}_y}{\phi_b M_{ny}\alpha_y} \le 1.0 \quad (Eq.\ C5.2.2\text{-}1)$$

$$\frac{18.4}{(0.85)(66.8)}+\frac{(1.0)(36.9)}{(0.95)(76.0)(0.867)} = 0.324 + 0.589 = 0.913 < 1.0 \quad \text{OK}$$

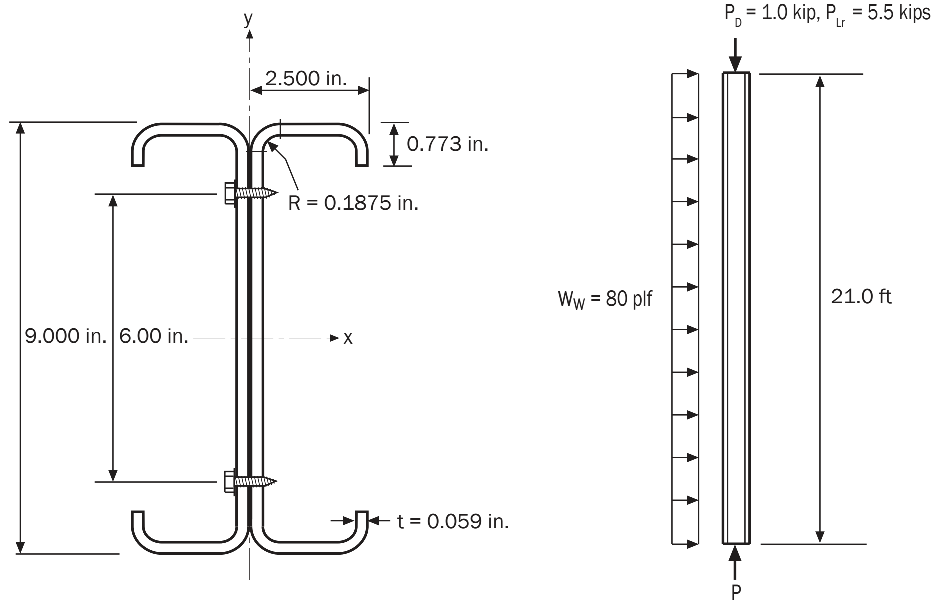
$$\frac{\overline{P}}{\phi_c P_{no}}+\frac{\overline{M}_x}{\phi_b M_{nx}}+\frac{\overline{M}_y}{\phi_b M_{ny}} \le 1.0 \qquad (\textit{Eq. } \text{C5.2.2-2})$$

P_{no} = 87.0 kips (calculated above)

$$\frac{18.4}{(0.85)(87.0)}+\frac{36.9}{(0.95)(76.0)} \le 1.0 \qquad (\textit{Eq. } \text{C5.2.2-2})$$

0.249 + 0.511 = 0.760 < 1.0 OK

Example III-9: I Section - Built-Up from Channels



Gross properties of each channel

t = 0.059 in.	I_{xi} = 10.3 in.^{4}	I_{yi} = 0.698 in.^{4}
A_i = 0.881 in.^{2}	r_{xi} = 3.42 in.	r_{yi} = 0.890 in.
C_{wi} = 11.9 in.^{6}	S_{xi} = 2.29 in.^{3}	$\overline{x}_i$ = 0.641 in.
J_i = 0.00102 in.^{4}	m_i = 1.05 in.	

Given:

1. Steel: F_y = 55 ksi, F_u = 70 ksi
2. Section: Two 9CS2.5x059 back to back as shown
3. Length: 21.0 ft
4. Braced for buckling about the x-axis at the ends only
5. Braced for buckling about the y-axis and for torsion at the ends and mid-span (10.5 feet)
6. $K_x = K_y = K_t = 1.0$
7. Sections connected by pairs of #10 screws at 36 in. on center spaced 6 inches apart along the y-axis of the channel sections

Required:

1. Check members for adequacy using:
   a. ASD - using ASCE/SEI 7-05 load combination D + 0.75W + 0.75L_r
   b. LRFD - using ASCE/SEI 7-05 load combination 1.2D + 1.6W + 0.5L_r

Solution:

Nominal Axial Strength, P_n (Section D1.2)

a) Properties of built-up section

$$A = 2A_i = (2)(0.881) = 1.76 \text{ in.}^2$$

$$I_x = 2I_{xi} = (2)(10.3) = 20.6 \text{ in.}^4$$

r_x = 3.42 in. (same as single section)

$$I_y = 2\left[I_{yi} + A_i\bar{x}_i^{\,2}\right]$$

$$= 2\left[0.698 + (0.881)(0.641)^2\right] = 2.12 \text{ in.}^4$$

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{2.12}{1.76}} = 1.10 \text{ in.}$$

$$C_w = 2C_{wi} = (2)(11.9) = 23.8 \text{ in.}^6$$

$$J = 2J_i = (2)(0.00102) = 0.00204 \text{ in.}^4$$

x_o = 0.0 (distance from shear center to centroid of combined shape)

$$r_o = \sqrt{r_x^{\,2} + r_y^{\,2} + x_o^{\,2}}$$

$$= \sqrt{3.42^2 + 1.10^2 + 0.0^2} = 3.59 \text{ in.}$$

b) X-axis flexural buckling per Section C4.1.1: The buckling mode does not involve relative deformations that produce shear forces in the connectors between individual shapes, so *Eq.* D1.2-1 does not apply.

$$\left(\frac{KL}{r}\right)_x = \frac{(1.0)(21.0)(12.0)}{3.42} = 73.7$$

$$F_e = \frac{\pi^2 E}{(KL/r)_x^{\,2}} \qquad (\textit{Eq. } \text{C4.1.1-1})$$

$$= \frac{\pi^2(29500)}{(73.7)^2} = 53.6 \text{ ksi}$$

c) Y-axis flexural buckling per Section C4.1.1: The buckling mode involves relative deformations that produce shear forces in the connectors between individual shapes, so *Eq.* D1.2-1 applies.

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \qquad (\textit{Eq. } \text{D1.2-1})$$

$$= \sqrt{\left(\frac{(1.0)(10.5)(12.0)}{1.10}\right)^2 + \left(\frac{36.0}{0.890}\right)^2} = 121.5$$

a/r_i = 36.0/0.890 = 40.4 < 121.5/2 OK

$$F_e = \frac{\pi^2 E}{(KL/r)_y^{\,2}} \qquad (\textit{Eq. } \text{C4.1.1-1})$$

$$= \frac{\pi^2(29500)}{(121.5)^2} = 19.7 \text{ ksi} \quad \leftarrow \text{CONTROLS}$$

d) Torsional buckling per Section C4.1.2: Since the section is doubly-symmetric,

$$F_e = \sigma_t = \frac{1}{Ar_o^{\,2}}\left[GJ + \frac{\pi^2 EC_w}{(K_tL_t)^2}\right] \qquad (\textit{Eq. } \text{C3.1.2.1-9})$$

$$F_e = \frac{1}{(1.76)(3.59)^2}\left[(11300)(0.00204)+\frac{\pi^2(29500)(23.8)}{\left[(1.0)(10.5)(12.0)\right]^2}\right] = 20.3 \text{ ksi}$$

e) Y-axis flexural buckling controls axial strength

F_e = 19.7 ksi

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \qquad (Eq.\ C4.1\text{-}4)$$

$$\lambda_c = \sqrt{\frac{55.0}{19.7}} = 1.67 > 1.5$$

$$F_n = \left[\frac{0.877}{\lambda_c^{\,2}}\right]F_y \qquad (Eq.\ C4.1\text{-}3)$$

$$F_n = \left[\frac{0.877}{(1.67)^2}\right]55 = 17.3 \text{ ksi}$$

It can be shown that at an axial compression stress of f = 17.3 ksi, the effective area of one channel is 0.620 in.^{2}. (calculations not shown); therefore,

$A_e = (2)(0.620) = 1.24\text{ in.}^2$

$P_n = A_eF_n$ (*Eq.* C4.1-1)

$= (1.24)(17.3) = 21.5\text{ kips}$

Nominal Flexural Strength, M_n

a) Check the maximum permitted screw spacing for the built-up section using the requirements of Section D1.1

$$s_{max} = L/6 \le \frac{2gT_s}{mq} \qquad (Eq.\ D1.1\text{-}1)$$

g = 6.0 in. (screw gage)

m = 1.05 in.

Screw tension design strength, T_s, is the smaller of the screw pull-out, pull-over or tension strengths.

Pull-out

$P_{not} = 0.85t_cdF_{u2}$ (*Eq.* E4.4.1-1)

$= (0.85)(0.059)(0.190)(70) = 0.667\text{ kips}$

Pull-over

Assuming no independent washer under the screw head, use Section E4.4.2(b)

$d'_w = d_h = 0.399$ in. (washer head diameter)

$P_{nov} = 1.5t_1d'_wF_{u1}$ (*Eq.* E4.4.2-1)

$= (1.5)(0.059)(0.399)(70) = 2.47\text{ kips}$

Screw tension

P_{ts} = 2.10 kips (from screw manufacturer)

$P_{nt} = P_{ts}$

$= 2.10$ kips

The nominal screw strength is the minimum of P_{not}, P_{nov} or P_{nt} = 0.667 kips.

ASD

$$T_s = \frac{0.667}{\Omega} = \frac{0.667}{3.0} = 0.222 \text{ kips}$$

LRFD

$$T_s = \phi 0.667 = (0.50)(0.667) = 0.334 \text{ kips}$$

The design load on the beam between fasteners, q, is taken as 3 times the uniformly distributed load.

$$q = (0.75)(3)\left(\frac{0.080}{12.0}\right) = 0.0150 \text{ kips per in. (ASD, using a wind load factor of 0.75)}$$

$$q = (1.6)(3)\left(\frac{0.080}{12.0}\right) = 0.0320 \text{ kips per inch (LRFD, using a wind load factor of 1.6)}$$

ASD

$$s_{max} = (21.0)(12.0)/6 \le \frac{(2)(6.0)(0.222)}{(1.05)(0.0150)} \qquad (Eq.\ D1.1\text{-}1)$$

$= 42.0$ in. ≤ 169 in.

LRFD

$$s_{max} = (21.0)(12.0)/6 \le \frac{(2)(6.0)(0.334)}{(1.05)(0.0320)} \qquad (Eq.\ D1.1\text{-}1)$$

$=42.0$ in. ≤ 119 in.

The spacing of 36 in. is OK for both ASD and LRFD, therefore section can be considered a built-up section in both cases.

b) Calculate the flexural strength according to Section C3.1.2.1.

$$F_e = \frac{C_b r_o A}{S_f}\sqrt{\sigma_{ey}\sigma_t} \qquad (Eq.\ C3.1.2.1\text{-}4)$$

$$\sigma_{ey} = \frac{\pi^2 E}{\left(K_y L_y / r_y\right)^2} \qquad (Eq.\ C3.1.2.1\text{-}8)$$

$$= \frac{\pi^2 (29500)}{\left[(1.0)(10.5)(12.0)/1.10\right]^2} = 22.2 \text{ ksi}$$

$\sigma_t = 20.3$ ksi (computed above)

$C_b = 1.0$

$$F_e = \frac{(1.0)(3.59)(1.76)}{(2)(2.29)}\sqrt{(22.2)(20.3)} \qquad (Eq.\ C3.1.2.1\text{-}4)$$

$= 29.3$ ksi

Since $F_e < 0.56F_y = (0.56)(55) = 30.8 \text{ ksi}$,

$$F_c = F_e = 29.3 \text{ ksi} \qquad (Eq.\ C3.1.2.1\text{-}3)$$

It can be shown that the section is fully effective at an extreme fiber flexural stress of f = 29.3 ksi (calculations not shown), therefore:

$S_c = S_{gross} = (2)(2.29) = 4.58\ \text{in.}^3$

$M_n = S_c F_c$ (*Eq.* C3.1.2.1-1)

$= (4.58)(29.3) = 134\ \text{kip-in.}$

Combined Compression and Bending

a) ASD - check according to Section C5.2.1

ASCE/SEI 7-05 ASD load combination D + 0.75W + 0.75L_r controls

Required strength

$P = P_D + 0.75P_{Lr}$

$= 1.0 + (0.75)(5.5) = 5.13\ \text{kips}$

$M_x = 0.75M_W = 0.75\dfrac{W_W L^2}{8}$

$= 0.75\dfrac{(0.080)(21.0)^2}{8} = 3.308\ \text{kip-ft} = 39.7\ \text{kip-in.}$

$M_y = 0$

Combined compression and bending

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{mx} M_x}{M_{nx}\alpha_x} + \frac{\Omega_b C_{my} M_y}{M_{ny}\alpha_y} \le 1.0 \qquad (\textit{Eq. C5.2.1-1})$$

$\Omega_c = 1.80$

$\Omega_b = 1.67$

$$P_{Ex} = \frac{\pi^2 E I_x}{(K_x L_x)^2} \qquad (\textit{Eq. C5.2.1-6})$$

$$= \frac{\pi^2 (29500)(20.6)}{[(1.0)(21.0)(12.0)]^2} = 94.4\ \text{kips}$$

$$\alpha_x = 1 - \frac{\Omega_c P}{P_{Ex}} > 0 \qquad (\textit{Eq. C5.2.1-4})$$

$$= 1 - \frac{(1.80)(5.13)}{94.4} = 0.902$$

$C_{mx} = 1.0$

$$\frac{(1.80)(5.13)}{21.5} + \frac{(1.67)(1.0)(39.7)}{(134)(0.902)} \le 1.0 \qquad (\textit{Eq. C5.2.1-1})$$

$0.429 + 0.549 = 0.978 < 1.0$ OK

$$\frac{\Omega_c P}{P_{no}} + \frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} \le 1.0 \qquad (\textit{Eq. C5.2.1-2})$$

$P_{no} = A_e F_y$

$= (2)(24.3) = 48.6\ \text{kips}$ (from Table III-1)

$$\frac{(1.80)(5.13)}{48.6}+\frac{(1.67)(39.7)}{134}\le 1.0 \qquad (Eq.\ C5.2.1\text{-}2)$$

0.190 + 0.495 = 0.685 < 1.0 OK

b) LRFD - check according to Section C5.2.2

ASCE/SEI 7-05 LRFD load combination 1.2D + 1.6W + 0.5L_r controls

Required strength

$$\overline{P} = P_u = 1.2P_D + 0.5P_{Lr}$$

$$= (1.2)(1.0)+(0.5)(5.5) = 3.95\ \text{kips}$$

$$\overline{M}_x = M_{ux} = 1.6\frac{W_W L^2}{8}$$

$$= 1.6\frac{(0.080)(21.0)^2}{8} = 7.056\ \text{kip-ft} = 84.7\ \text{kip-in.}$$

$$\overline{M}_y = 0$$

Combined Compression and Bending

$$\frac{\overline{P}}{\phi_c P_n}+\frac{C_{mx}\overline{M}_x}{\phi_b M_{nx}\alpha_x}+\frac{C_{my}\overline{M}_y}{\phi_b M_{ny}\alpha_y}\le 1.0 \qquad (Eq.\ C5.2.2\text{-}1)$$

ϕ_c = 0.85

ϕ_b = 0.90

$$P_{Ex} = \frac{\pi^2 EI_x}{(K_x L_x)^2} \qquad (Eq.\ C5.2.2\text{-}6)$$

$$P_{Ex} = \frac{\pi^2(29500)(20.6)}{[(1.0)(21.0)(12.0)]^2} = 94.4\ \text{kips}$$

$$\alpha_x = 1-\frac{\overline{P}}{P_{Ex}} \qquad (Eq.\ C5.2.2\text{-}4)$$

$$= 1-\frac{3.95}{94.4} = 0.958$$

C_{mx} = 1.0

$$\frac{3.95}{(0.85)(21.5)}+\frac{(1.0)(84.7)}{(0.90)(134)(0.958)}\le 1.0 \qquad (Eq.\ C5.2.2\text{-}1)$$

$0.216+0.733=0.949<1.0$ OK

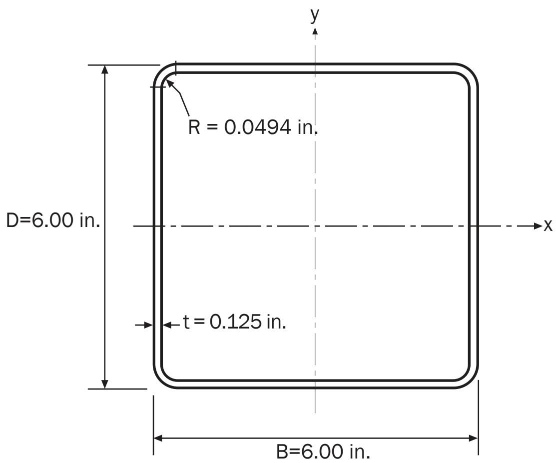
$$\frac{\overline{P}}{\phi_c P_{no}}+\frac{\overline{M}_x}{\phi_b M_{nx}}+\frac{\overline{M}_y}{\phi_b M_{ny}}\le 1.0 \qquad (Eq.\ C5.2.2\text{-}2)$$

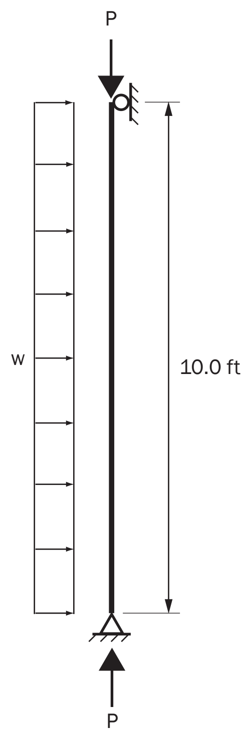
from above, P_{no} = 48.6 kips

$$\frac{3.95}{(0.85)(48.6)}+\frac{84.7}{(0.90)(134)}\le 1.0 \qquad (Eq.\ C5.2.2\text{-}2)$$

0.096+ 0.702 = 0.798 < 1.0 OK

Example III-10: Square HSS Section – Bending and Compression





Given:

1. Steel: F_y = 46 ksi, F_u = 65 ksi
2. Section: HSS 6x6x⅛ (See Table 1-12, 2005 AISC Steel Construction Manual)[4]
3. Simply supported at both ends
4. Braced about both x- and y-axis at both ends
5. $K_x = K_y = 1.0$, $L_x = L_y$ = 10.0 ft
6. Dead Load: P_D = 7.50 kips

   Live Load: P_L = 37.5 kips, w_L = 0.100 kips/ft

Required:

1. Determine the ASD allowable axial strength, P_n/Ω_c
2. Determine the LRFD design axial strength, $\phi_c P_n$
3. Compare the available strengths to those calculated per the 2005 AISC Specification[5]. The inside radius given above is selected to give the same flat width, w, for the flanges and webs used in the AISC calculations.
4. Verify the combined bending and compression strength of the section for the following ASCE/SEI 7-05 load combinations:
   a. ASD: D + L
   b. LRFD: 1.2D + 1.6L

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[4] AISC, Steel Construction Manual – 13th Edition, American Institute of Steel Construction, Chicago, IL, 2005

[5] AISC 360, Specification for Structural Steel Buildings, American Institute of Steel Construction, Chicago, IL, 2005

Solution:

Nominal Axial Strength, P_n (Section C4.1)

Since the square tube is a doubly symmetrical closed section, it is not subject to flexural-torsional or distortional buckling. The nominal axial strength, P_n, can be computed according to *Specification* Sections C4.1(a) and C4.1.1.

a) Calculate the section properties of gross section using *Manual* Section 3.2, Properties of Line Elements.

i. Corner line elements (from Case 1 of Manual Section 3.2.2)

$$r = R + t/2 = 0.0494 + 0.125/2 = 0.112 \text{ in.}$$

$$l = 1.57r = (1.57)(0.112) = 0.176 \text{ in.}$$

$$c = 0.637r = (0.637)(0.112) = 0.0713 \text{ in.}$$

$$I_1 = 0.149\, r^3 = (0.149)(0.112)^3 = 0.000209 \text{ in.}^3$$

ii. Stiffened flange and web elements

$$w = B - 2(R + t) = 6.00 - 2(0.0494 + 0.125) = 5.651 \text{ in.}$$

ii. Gross section properties

$$A = 4(w + l)t = 4(5.651 + 0.176)(0.125) = 2.914 \text{ in.}^2$$

$$I_x = I_y = \left\{2\left[\frac{w^3}{12} + w\left(\frac{D}{2} - \frac{t}{2}\right)^2\right] + 4\left[I_1 + l\left(\frac{D}{2} - \frac{t}{2} - r + c\right)^2\right]\right\}t$$

$$= \left\{\begin{array}{l} 2\left[\dfrac{5.651^3}{12} + 5.651\left(3.00 - \dfrac{0.125}{2}\right)^2\right] \\ +4\left[0.000209 + 0.176\left(3.00 - \dfrac{0.125}{2} - 0.112 + 0.0713\right)^2\right]\end{array}\right\}0.125$$

$$= 16.69 \text{ in.}^4$$

$$r_x = r_y = \sqrt{16.69/2.914} = 2.393 \text{ in.}$$

b) Calculate the nominal flexural buckling stress, F_n.

$$\frac{K_xL_x}{r_x} = \frac{K_yL_y}{r_y} = \frac{(1.0)(10.0)(12)}{2.393} = 50.15$$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 29500}{(50.15)^2} = 115.8 \text{ ksi} \qquad (Eq.\ C4.1.1\text{-}1)$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} = \sqrt{\frac{46}{115.8}} = 0.630 < 1.5 \qquad (Eq.\ C4.1\text{-}4)$$

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y = \left(0.658^{0.630^2}\right)46 = 38.96 \text{ ksi} \qquad (Eq.\ C4.1\text{-}2)$$

c) Calculate the effective area, A_e.

For stiffened flat elements

$w/t = 5.651/0.125 = 45.2$

$k = 4.0$

$$F_{cr} = k\frac{\pi^2 E}{12(1-\mu)^2}\left(\frac{t}{w}\right)^2 \qquad (Eq.\ B2.1\text{-}5)$$

$$= 4.0\frac{\pi^2(29500)}{12(1-0.3)^2}\left(\frac{1}{45.2}\right)^2 = 52.20 \text{ ksi}$$

$\lambda = \sqrt{\frac{F_n}{F_{cr}}} = \sqrt{\frac{38.96}{52.20}} = 0.864 > 0.673$; therefore, flat elements are not fully effective.(*Eq.* B2.1-4)

$$\rho = (1-0.22/\lambda)/\lambda \qquad (Eq.\ B2.1\text{-}3)$$

$$= (1-0.22/0.864)/0.864 = 0.863$$

$$b = \rho w = (0.863)(5.651) = 4.877 \text{ in.} \qquad (Eq.\ B2.1\text{-}2)$$

$$A_e = A - 4(w-b)t$$

$$= 2.914 - 4(5.651-4.877)(0.125) = 2.527 \text{ in.}^2$$

d) Calculate the nominal axial strength, P_n.

$$P_n = A_e F_n = 2.527(38.96) = 98.5 \text{ kips} \qquad (Eq.\ C4.1\text{-}1)$$

Available Axial Strengths

ASD

$$P \le \frac{P_n}{\Omega_c} \qquad (Eq.\ A4.1.1\text{-}1)$$

$$\Omega_c = 1.80$$

$$P \le \frac{98.5}{1.80} = 54.7 \text{ kips}$$

LRFD

$$P_u \le \phi_c P_n \qquad (Eq.\ A5.1.1\text{-}1)$$

$$\phi_c = 0.85$$

$$P_u \le 0.85(98.5) = 83.7 \text{ kips}$$

Comparison between available axial strengths computed using AISI and AISC Specifications

The available strengths determined in Part 2 and 3 above are based on the AISI *Specification*. These values can be compared with those determined using the 2005 AISC Specification, as listed in Table 4-4 of the 2005 AISC Steel Construction Manual

Available Strength	AISI (kips)	AISC (kips)	$\frac{\text{AISI}}{\text{AISC}}$
ASD: P_n/Ω_c	54.7	54.3	1.01
LRFD: $\phi_c P_n$	83.7	81.6	1.03

The above comparison shows that the available strengths based on the AISI and AISC specifications are practically the same, even though the design wall thickness, the design equations, and the safety and resistance factors differ as follows:

a) The AISI *Specification* uses the nominal wall thickness of 0.125 in., while the AISC Specification requires the use of a design wall thickness of 93% of the nominal thickness = 0.116 in.

b) The design equations for computing the nominal axial strengths differ. The AISI *Specification* uses the effective area multiplied by the nominal column buckling stress, while the AISC Specification uses the gross area multiplied by a reduced column buckling stress, using QF_y to replace F_y.

c) For the AISI *Specification*, Ω_c = 1.80 and ϕ_c = 0.85. For the AISC Specification, Ω_c = 1.67 and ϕ_c = 0.90.

d) The differences shown in the table above apply only to the HSS 6x6x⅛ with F_y = 46 ksi. For other sections and yield stresses, the AISI/AISC strength ratios may be slightly different.

Check combined compression and bending – ASD (Section C5.2.1)

a) Required strength

$$M_x = M_{live} = \frac{wL^2}{8} = \frac{(0.100)(10.0)^2}{8} = 1.25 \text{ kip-ft}$$

$$P = P_{dead} + P_{live} = 7.50 + 37.5 = 45.0 \text{ kips}$$

b) From Part (1) above, P_n = 98.5 kips

$$\frac{\Omega_c P}{P_n} = \frac{1.80(45.0)}{98.5} = 0.822 > 0.15$$; therefore, use *Eqs*. C5.2.1-1 and C5.2.1-2

C_{mx} = 1.0

$$P_{Ex} = \frac{\pi^2 EI_x}{(K_x L_x)^2} \qquad (\textit{Eq. C5.2.1-6})$$

$$= \frac{\pi^2(29500)(16.69)}{[(1.0)(120.0)]^2} = 337 \text{ kips}$$

$$\alpha_x = 1 - \frac{\Omega_c P}{P_{Ex}} > 0 \qquad (\textit{Eq. C5.2.1-4})$$

$$= 1 - \frac{(1.80)(45.0)}{337} = 0.760$$

M_y = 0.0

$$\frac{\Omega_c P}{P_n} + \frac{\Omega_b C_{mx} M_x}{M_{nx}\alpha_x} + \frac{\Omega_b C_{my} M_y}{M_{ny}\alpha_y} \le 1.0 \qquad (\textit{Eq. C5.2.1-1})$$

From Example II-8;

M_{nx} = 19.0 kip-ft

$$\frac{(1.80)(45.0)}{98.5} + \frac{(1.67)(1.0)(1.25)}{(19.0)(0.760)} = 0.967 < 1.0 \quad \text{OK}$$

$$\frac{\Omega_c P}{P_{no}} + \frac{\Omega_b M_x}{M_{nx}} + \frac{\Omega_b M_y}{M_{ny}} \le 1.0 \quad (Eq.\ C5.2.1\text{-}2)$$

Computation of P_{no}

From Example II-8, for f = F_y = 46 ksi, b = 4.608 in.

$$A_e = A - 4(w - b)t$$

$$= 2.914 - 4(5.651 - 4.608)(0.125) = 2.393 \text{ in.}^2$$

$$P_{no} = A_e F_n = A_e F_y = (2.393)(46) = 110 \text{ kips}$$

$$\frac{(1.80)(45.0)}{110} + \frac{(1.67)(1.25)}{19.0} = 0.846 < 1.0 \quad \text{OK} \quad (Eq.\ C5.2.1\text{-}2)$$

Check combined compression and bending – LRFD (Section C5.2.2)

a) Required strength

$$\overline{M}_x = M_{ux} = \frac{1.6 w_{live} L^2}{8} = \frac{1.6(0.100)(10.0)^2}{8} = 2.00 \text{ kip-ft}$$

$$\overline{P} = P_u = 1.2P_{dead} + 1.6P_{live} = 1.2(7.50) + 1.6(37.5) = 69.0 \text{ kips}$$

a) From Part (1) above, P_n = 98.5 kips

$$\frac{\overline{P}}{\phi_c P_n} = \frac{69.0}{(0.85)(98.5)} = 0.824 > 0.15$$; therefore, use *Eqs*. C5.2.2-1 and C5.2.2-2

C_{mx} = 1.0

$$P_{Ex} = \frac{\pi^2 E I_x}{(K_x L_x)^2} \quad (Eq.\ C5.2.2\text{-}6)$$

$$= \frac{\pi^2 (29500)(16.69)}{\left[(1.0)(120.0)\right]^2} = 337 \text{ kips}$$

$$\alpha_x = 1 - \frac{P}{\phi_c P_{Ex}} > 0 \quad (Eq.\ C5.2.2\text{-}4)$$

$$= 1 - \frac{69.0}{0.85(337)} = 0.760$$

$\overline{M}_y$ = 0.0

$$\frac{\overline{P}}{\phi_c P_n} + \frac{C_{mx}\overline{M}_x}{\phi_b M_{nx}\alpha_x} + \frac{C_{my}\overline{M}_y}{\phi_b M_{ny}\alpha_y} \le 1.0 \quad (Eq.\ C5.2.2\text{-}1)$$

From Example II-8;

M_{nx} = 19.0 kip-ft

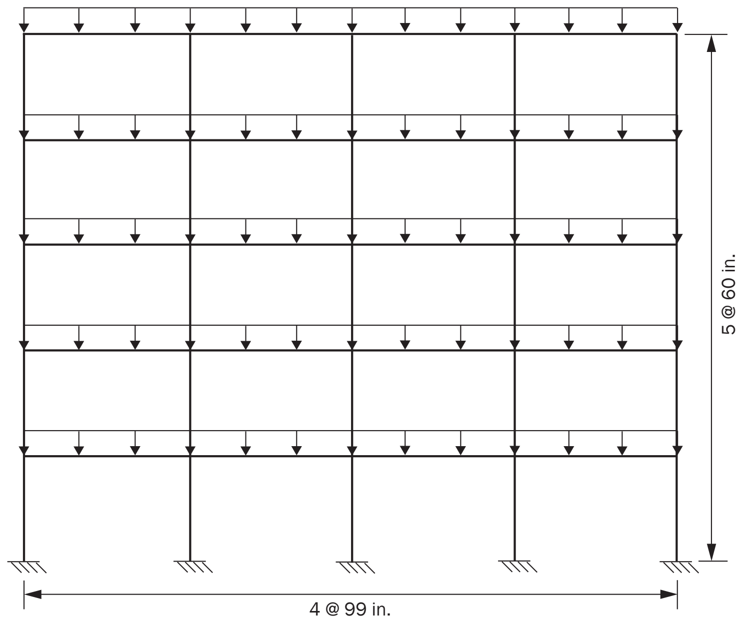
$$\frac{69.0}{(0.85)(98.5)} + \frac{(1.0)(2.0)}{(0.95)(19.0)(0.760)} = 0.970 < 1.0 \quad \text{OK}$$

$$\frac{\overline{P}}{\phi_c P_{no}} + \frac{\overline{M}_x}{\phi_b M_{nx}} + \frac{\overline{M}_y}{\phi_b M_{ny}} \le 1.0 \quad (Eq.\ C5.2.2\text{-}2)$$

P_{no} = 110 kips from Part 4 above

$$\frac{69.0}{(0.85)(110)} + \frac{2.0}{(0.95)(19.0)} = 0.849 < 1.0 \quad \text{OK} \qquad (\textit{Eq. } \text{C5.2.2-2})$$

Example III-11: Frame Design by Second Order Analysis



Given:

1. Steel: F_y = 55 ksi
2. Frame as shown above
3. All beams uniformly loaded with 35.7 lbs/in. (LRFD factored loading)
4. Column properties

   A_g = 0.936 in.^{2}

   I_x = I_y = 1.27 in.^{4}

   r_x = r_y = 1.16 in.

   S_e = 0.847 in.^{3}
5. Beam properties

   A_g = 0.780 in.^{2}

   I_x = 1.70 in.^{4}
6. Columns are not subject to local, distortional or flexural-torsional buckling.
7. Connections are semi-rigid with the following nominal rigidities:

   Beam ends: 750 kip-in./radian

   Column bases: 3000 kip-in./radian

Required:

1. Check the adequacy of the typical interior column at the ground level using the provisions of Appendix 2 *Second-Order Analysis* with LRFD.
2. Check the adequacy of the typical interior column at the ground level using the rational effective length procedure with LRFD.

Solution:

1. Second-Order Analysis Procedure

A second-order analysis is required using 1) reduced stiffness to account for the effects of inelasticity, and 2) notional loads to account for structure out-of-plumbness. Under these conditions, the effective length factor in the plane of the frame, K_x, may be taken as 1.0.

a) Compute the reduced axial and flexural stiffnesses per *Specification* Appendix 2, Section 2.2.3

Although the *Specification* requires member stiffness reductions only for "members whose axial and flexural stiffness are considered to contribute to the lateral stability of the structure", as a practical matter, reducing the stiffness of all members in the computer analysis is much more convenient. The required modified modulus of elasticity is:

$$E^* = 0.8\tau_b E \qquad (Eq.\ 2\text{-}1)$$

where

$$\tau_b = 1.0 \text{ for } \alpha P_r/P_y \le 0.5$$

$$= 4\left[\alpha P_r/P_y\left(1-\alpha P_r/P_y\right)\right] \text{ for } \alpha P_r/P_y > 0.5$$

Assume $\alpha P_r/P_y \le 0.5$ for all members; therefore $\tau_b = 1.0$. Confirm this assumption after the analysis.

$$E^* = 0.8(1.0)29500 = 23600 \text{ ksi} \qquad (Eq.\ 2\text{-}1)$$

b) Compute the reduced connection stiffnesses per *Specification* Appendix 2, Section 2.2.3

Reduce the nominal connection stiffnesses in the computer model to 80% of their nominal value as required by Section 2.2.3.

Column bases:

$$K_{col} = 0.8(3000) = 2400 \text{ kip-in./radian}$$

Beam ends:

$$K_{beam} = 0.8(750) = 600 \text{ kip-in./radian}$$

c) Compute the notional loads per *Specification* Appendix 2, Section 2.2.4

Notional loads are lateral loads applied in the second-order analysis at each level to account for the effects of out-of-plumbness. Alternatively, the structure can be modeled with the out-of-plumb geometry. Unless the analysis software includes provisions for automatically generating the out-of-plumb geometry, the use of notional loads is usually simpler. The required notional load at each beam level is calculated as:

$$N_i = (1/240)Y_i$$

The gravity load, Y_i, at each level is:

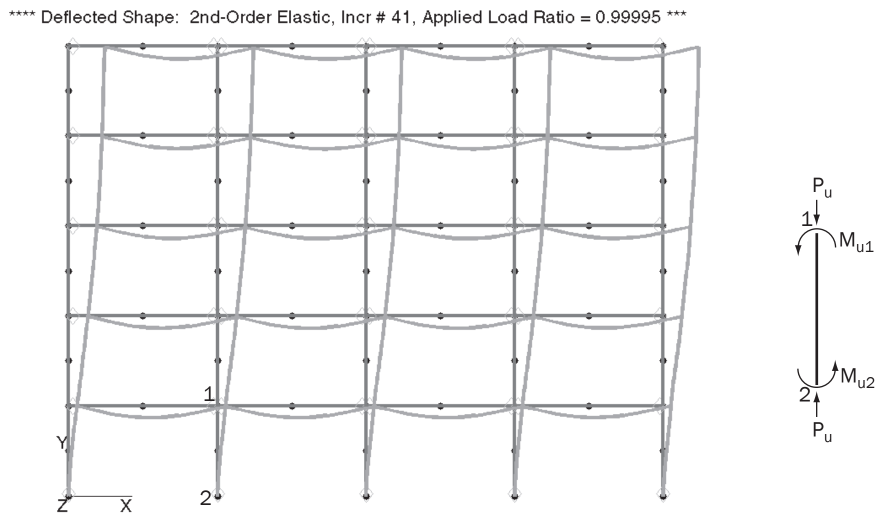
$$Y_i = 35.7(4)(99.0) = 14{,}100 \text{ lbs}$$

The notional load, N_i, at each level is:

$$N_i = (1/240)14{,}100 = 58.8 \text{ lbs}$$

Using the specified gravity and notional loading and the reduced stiffness, the frame is analyzed using MASTAN2[6]. The resulting deformed shape is shown below.

[6] Ziemian, R.D. and McGuire, W, *MASTAN2*, available at http://www.mastan2.com



The resulting axial force and moments in the most heavily loaded interior column are found to be:

P_u = 17.7 kips

M_{u1} = 8.88 kip-in. at the bottom of the column

M_{u2} = 3.55 kip-in. at the top of the column

2. Second-Order Combined Strength Check

a) Check the assumption that $P_r/P_y \leq 0.5$. For the member with the largest axial load:

$P_r = P_u = 17.7$ kips

$$\frac{P_r}{P_y} = \frac{17.7}{(0.936)(55)} = 0.344 < 0.5$$; therefore, assumption that $\tau_b = 1.0$ is OK.

b) Calculate the axial strength.

Since the column is not subject to local, distortional or flexural-torsional buckling, the strength is governed by flexural column buckling.

$$K_xL_x = K_yL_y = (1.0)(60.0) = 60.0 \text{ in.}$$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} \quad (Eq.\ C4.1.1\text{-}1)$$

$$= \frac{\pi^2 29500}{(60.0/1.16)^2} = 109 \text{ ksi}$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}} \quad (Eq.\ C4.1\text{-}4)$$

$$= \sqrt{\frac{55}{109}} = 0.710 < 1.5 \text{; therefore,}$$

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \quad (Eq.\ C4.1\text{-}2)$$

$$= \left(0.658^{0.710^2}\right)55 = 44.5 \text{ ksi}$$

$P_n = A_eF_n$ (*Eq.* C4.1-1)

$= (0.936)(44.5) = 41.7$ kips

c) Calculate the flexural strength.

Since the column is not subject to local, distortional or lateral-torsional buckling, the flexural yield strength governs.

$M_n = S_eF_y$ (*Eq.* C3.1.1-1)

$= (0.847)(55) = 46.6$ kip-in.

d) Check the combined strength.

Since the required moments were determined by a second-order analysis in accordance with Appendix 2, Section 2.1, C_m and α are both taken as 1.0; therefore, *Eq.* C5.2.2-1 will govern.

$\overline{P} = P_u = 17.7$ kips

$\overline{M} = M_u = 8.88$ kip-in.

$\phi_c = 0.85$

$\phi_b = 0.95$

$$\frac{\overline{P}}{\phi_c P_n} + \frac{\overline{M}_x}{\phi_b M_{nx}} + \frac{\overline{M}_y}{\phi_b M_{ny}} \le 1.0$$ (*Eq.* C5.2.2-2)

$$\frac{17.7}{0.85(41.7)} + \frac{8.88}{0.95(46.6)} = 0.700 \le 1.0 \text{ OK}$$

3. Effective Length Analysis

In the traditional effective length approach, in-plane effective length factors, K_x, are determined by some means, often through the use of "alignment charts", such as those published by AISC in the Steel Construction Manual[7]. In this case, the presence of multiple levels of semi-rigid connections complicates this approach. Alternatively, the elastic buckling stress, F_e, can be directly determined by an elastic buckling analysis of the entire structure.

Using nominal member and connection stiffnesses and no notional loads, the smallest elastic buckling load of the structure was determined to be 1.75 times the factored load using the "elastic critical load" analysis of MASTAN2. For column 1-2, the axial load in the column under gravity load, P_u, is found to be 17.7 kips by a separate first-order analysis. The elastic buckling stress of the column in the plane of the frame, F_{ex}, can then be calculated as:

$$F_{ex} = 1.75\frac{P_u}{A} = 1.75\left(\frac{17.7}{0.936}\right) = 33.1 \text{ ksi}$$

$$\lambda_c = \sqrt{\frac{F_y}{F_e}}$$ (*Eq.* C4.1-4)

$$= \sqrt{\frac{55}{33.1}} = 1.29 < 1.5 \text{; therefore}$$

[7] AISC, Steel Construction Manual – 13th Edition, American Institute of Steel Construction Inc., Chicago, IL 2005

$$F_n = \left(0.658^{\lambda_c^2}\right)F_y \qquad (Eq.\ C4.1\text{-}2)$$

$$= \left(0.658^{1.29^2}\right)55 = 27.4\ \text{ksi}$$

$$P_n = A_eF_n \qquad (Eq.\ C4.1\text{-}1)$$

$$= (0.936)(27.4) = 25.6\ \text{kips}$$

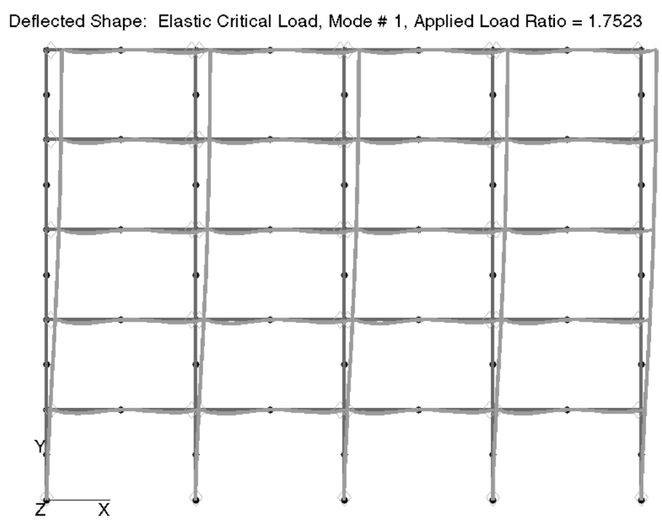
Since there are no significant bending moments in the interior columns under this loading in the first-order analysis, an axial strength check is sufficient.

$P_u = 17.7$ kips

$\phi_c = 0.85$

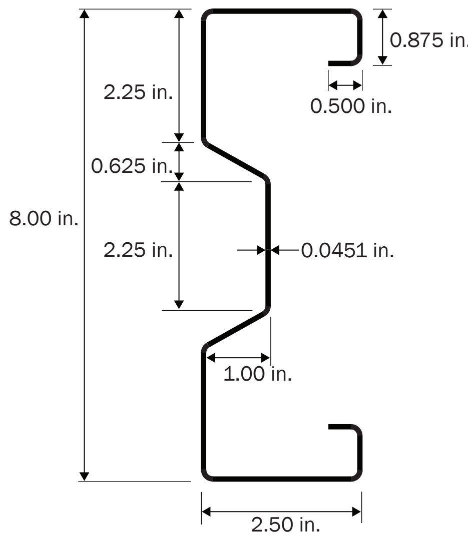
$$\frac{P_u}{\phi_c P_n} \le 1.0 \qquad (Eq.\ A5.1.1\text{-}1)$$

$$\frac{17.7}{0.85(25.6)} = 0.813 \le 1.0 \ \text{OK}$$



Both methods show that the lower columns of the frame are acceptable, but the 2nd order analysis is less conservative in this case.

Example III-12: Web-Stiffened C- Section by the Direct Strength Method - Compression



Given:

1. Steel: F_y = 50 ksi, F_u = 65 ksi
2. Sigma section (C-section with web stiffener) as shown above

Required:

Calculate the ASD and LRFD available compression strengths using the Direct Strength procedure from *Specification* Appendix 1. Consider the two cases of:

1) Continuously braced against flexural, flexural-torsional and distortional buckling
2) Discretely braced against flexural, flexural-torsional and distortional buckling at a spacing of 66.0 in.

Solution:

Although the Direct Strength method may be used for any cross-section, it is particularly well suited to this example, since the cross-section is somewhat complex and the *Specification* has no provisions applicable to the complex edge stiffeners on the flanges.

1. Perform a finite strip analysis

A finite strip analysis of the cross-section is performed using a program such as CUFSM[8]. A pure axial stress distribution is assumed with the fibers at F_y in compression. Results from a CUFSM analysis include the axial strength under the assumed stress distribution, P_y, and a graph of the section buckling strength versus unbraced length, shown below. Examination of the mode shape for the member at a length of 66 in. shows both lateral translation associated with flexural buckling and distortion of the cross section associated with distortional buckling; consequently, the elastic buckling load at this length is used for the distortional buckling limit state check. The dashed line superimposed on the right half of the graph represents the global buckling mode isolated from other limit states. The elastic buckling load at this length from this

[8] Schafer, B.W., Ádány, S. "Buckling analysis of cold-formed steel members using CUFSM: conventional and constrained finite strip methods." Eighteenth International Specialty Conference on Cold-Formed Steel Structures, Orlando, FL. October 2006. Available at www.ce.jhu.edu/bschafer/cufsm

line is used for the global buckling limit state check below. The critical buckling strengths are obtained by multiplying the yield strength, P_y, by the corresponding load factors obtained from the finite strip analysis.

From the analysis:

Yield strength

$$P_y = 37.4 \text{ kips}$$

Critical elastic local buckling strength

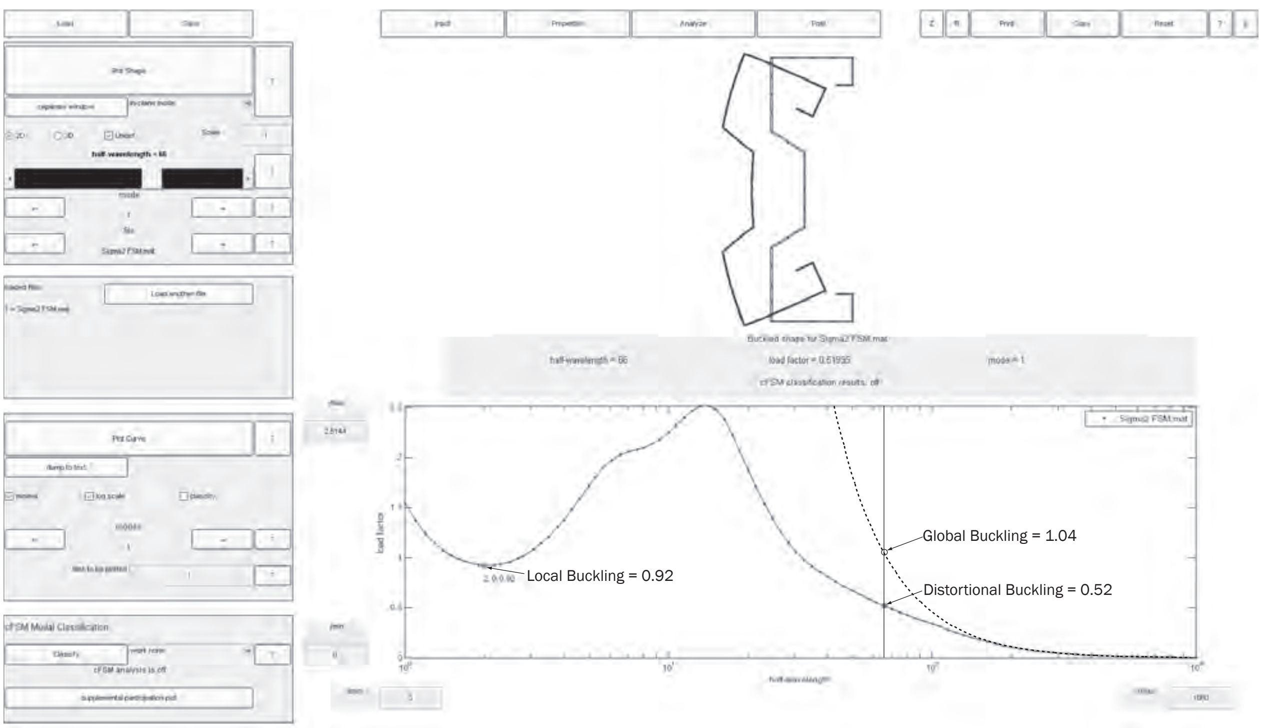
$$P_{cr\ell} = 0.92P_y = (0.92)(37.4) = 34.4 \text{ kips}$$

Critical elastic flexural buckling strength at 66.0 in.

$$P_{cre} = 1.04P_y = (1.04)(37.4) = 38.9 \text{ kips}$$

Critical elastic distortional buckling strength at 66.0 in.

$$P_{crd} = 0.52P_y = (0.52)(37.4) = 19.4 \text{ kips}$$



2. Calculate the nominal axial strength

Per section 1.2.1 of Appendix 1, take P_n as the lowest of the nominal strengths for flexural, torsional, or flexural-torsional buckling, P_{ne}, local buckling, $P_{n\ell}$ and distortional buckling, P_{nd}.

Case 1: The member is fully braced against global buckling and distortional buckling

1) Global buckling: The member is fully braced against global buckling; therefore,
$P_{ne} = P_y = 37.4$ kips

2) Local buckling:

$$\lambda_\ell = \sqrt{P_{ne}/P_{cr\ell}} \qquad (Eq.\ 1.2.1\text{-}7)$$

$$= \sqrt{37.4/34.4} = 1.04$$

Since $\lambda_\ell > 0.776$,

$$P_{n\ell} = \left(1-0.15\left(\frac{P_{cr\ell}}{P_{ne}}\right)^{0.4}\right)\left(\frac{P_{cr\ell}}{P_{ne}}\right)^{0.4} P_{ne} \qquad (Eq.\ 1.2.1\text{-}6)$$

$$= \left(1-0.15\left(\frac{34.4}{37.4}\right)^{0.4}\right)\left(\frac{34.4}{37.4}\right)^{0.4} 37.4 = 30.9 \text{ kips}$$

3) Distortional buckling: The member is fully braced against distortional buckling; therefore,

$P_{nd} = P_y = 37.4$ kips

Case 2: The member is discretely braced against global buckling and distortional buckling at 66.0 in. on center.

1) Global buckling: From the finite strip analysis, at 66.0 in.,

$$\lambda_c = \sqrt{P_y/P_{cre}} \qquad (Eq.\ 1.2.1\text{-}3)$$

$$= \sqrt{37.4/38.9} = 0.981$$

Since $\lambda_c < 1.5$,

$$P_{ne} = \left(0.658^{\lambda_c^2}\right)P_y \qquad (Eq.\ 1.2.1\text{-}1)$$

$$= \left(0.658^{0.981^2}\right)37.4 = 25.0 \text{ kips}$$

2) Local buckling:

$$\lambda_\ell = \sqrt{P_{ne}/P_{cr\ell}} \qquad (Eq.\ 1.2.1\text{-}7)$$

$$= \sqrt{25.0/34.4} = 0.852$$

Since $\lambda_\ell > 0.776$,

$$P_{n\ell} = \left(1-0.15\left(\frac{P_{cr\ell}}{P_{ne}}\right)^{0.4}\right)\left(\frac{P_{cr\ell}}{P_{ne}}\right)^{0.4} P_{ne} \qquad (Eq.\ 1.2.1\text{-}6)$$

$$= \left(1-0.15\left(\frac{34.4}{25.0}\right)^{0.4}\right)\left(\frac{34.4}{25.0}\right)^{0.4} 25.0 = 23.6 \text{ kips}$$

3) Distortional buckling:

$$\lambda_d = \sqrt{P_y/P_{crd}} \qquad (Eq.\ 1.2.1\text{-}10)$$

$$= \sqrt{37.4/19.4} = 1.39$$

Since $\lambda_d > 0.561$,

$$P_{nd} = \left(1 - 0.25\left(\frac{P_{crd}}{P_y}\right)^{0.6}\right)\left(\frac{P_{crd}}{P_y}\right)^{0.6} P_y \qquad (Eq.\ 1.2.1\text{-}9)$$

$$= \left(1 - 0.25\left(\frac{19.4}{37.4}\right)^{0.6}\right)\left(\frac{19.4}{37.4}\right)^{0.6} 37.4 = 21.0\ \text{kips}$$

4) The nominal axial strengths are therefore 30.9 kips for Case 1 (braced against global and distortional buckling) and 21.0 kips for Case 2 (braced at 66.0 in on center), governed by distortional buckling.

3. Calculate the available strengths

Check the limitations for prequalified beams in Table 1.1.1-1 to determine the appropriate safety and resistance factors. Since there is no prequalified category for C-sections with web stiffeners and complex lips, use the safety and resistance factors from Section A1.2(b)

Case 1: Continuously braced against global and distortional buckling

ASD - Allowable strength

$$\frac{P_n}{\Omega} = \frac{30.9}{2.00} = 15.5\ \text{kips} \qquad (\text{Eq. A4.1.1-1})$$

LRFD - Design strength

$$\phi P_n = 0.80(30.9) = 24.7\ \text{kips} \qquad (Eq.\ \text{A5.1.1-1})$$

Case 2: Discretely braced against global and distortional buckling at 66.0 in. on center

ASD - Allowable strength

$$\frac{P_n}{\Omega} = \frac{21.0}{2.00} = 10.5\ \text{kips} \qquad (Eq.\ \text{A4.1.1-1})$$

LRFD - Design strength

$$\phi P_n = 0.80(21.0) = 16.8\ \text{kips} \qquad (Eq.\ \text{A5.1.1-1})$$

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PART IV
CONNECTION DESIGN

For Use With the
2007 Edition of the
North American Specification for the Design
of Cold-Formed Steel Structural Members

SECTION 1 - WELDS

The application of welded connections must comply with the requirements set forth in Section E2 of the *Specification*. The *Specification* applies to the welding of parts where the thinnest part is 3⁄16 in. or less. For welded connections in which the thickness of the thinnest connected part is greater than 3⁄16 in., refer to the AISC specifications[1]. Welds shall be made in accordance with AWS D1.3, except resistance welds which shall be in accordance with AWS C1.3.

The provisions governing welds are organized by weld type in Sections E2.1 through E2.6. With the exception of resistance spot welds, the welded connections are subject to the limit states of:

1. Base metal rupture
2. Weld metal rupture

These must be separately checked and the lower of the two strengths is used. For welded connections in which not all elements of the cross-section are used to transmit force, Section E2.7 requires the consideration of shear lag in the member.

1.1 Notes On The Tables

Shown in Table IV-1 are the nominal shear strengths for unit length fillet welds made on various sheet thicknesses and for sheet tensile strengths of 45 ksi and 65 ksi. The nominal weld shear strength is found by interpolating between values in the Table, then multiplying by the length of fillet weld used (adding values for longitudinal plus transverse welds). For ASD, the weld design shear strength is determined by dividing the nominal weld shear strength by Ω. For LRFD, the weld design shear strength is found by multiplying the nominal weld shear strength by ϕ.

Nominal shear strengths of resistance welds, "spot welds", are provided in Table IV-2. For ASD, the weld allowable shear strength is determined by dividing the nominal weld shear strength by Ω. For LRFD, the weld design shear strength is found by multiplying the nominal weld shear strength by ϕ.

Table IV-3 gives the nominal shear strengths for sheets welded to thicker supporting members with 1/2 in., 5/8 in., and 3/4 in. arc spot welds, based on sheet strength. Values are provided for sheet tensile strengths of 45 ksi and 65 ksi. Nominal strengths are determined by interpolation based on the total sheet thickness being welded to the supporting structure. For ASD, the weld allowable shear strength is determined by dividing the nominal weld shear strength by Ω. For LRFD, the weld design shear strength is found by multiplying the nominal weld shear strength by ϕ. The strength of the weld metal must also be checked using *Specification Eq*. E2.2.1.2-1.

Table IV-4 gives the nominal shear strengths for a sheet welded to an identical sheet with 1/2 in., 5/8 in., and 3/4 in. arc spot welds, based on sheet strength. Values are provided for sheet tensile strengths of 45 ksi. Nominal strengths are determined by interpolation based on the thickness of one of the two identical sheets. For ASD, the weld allowable shear strength is determined by dividing the nominal weld shear strength by Ω. For LRFD, the weld design shear strength is found by multiplying the nominal weld shear strength by ϕ. The strength of the weld metal, F_{xx}, must exceed 45 ksi.

Table IV-5 gives the nominal tensile strengths for concentrically loaded 1/2 in., 5/8 in., and 3/4 in. arc spot welds based on sheet strength. Values are provided for sheet tensile strengths

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[1] AISC-360, *Specification for Structural Steel Buildings*, American Institute of Steel Construction, Chicago, IL, 2005

of 45 ksi and 65 ksi. Nominal strengths are determined by interpolation based on the total sheet thickness being welded to the supporting structure. When used as side lap connectors within a deck system, these values must be reduced 30 percent. In other eccentric connections, these values must be reduced by 50 percent. For ASD, the weld allowable shear strength is determined by dividing the nominal weld shear strength by Ω. For LRFD, the weld design shear strength is found by multiplying the nominal weld shear strength by ϕ. The strength of the weld metal must also be checked using *Specification Eq*. E2.2.2-1.

1.2 Welded Connection Design Tables

Table IV - 1

Fillet Welds
Shear of Sheet

Ω = See Table
ϕ = See Table

Nominal Shear Strength of a 1 in. long weld, P'_n, kips/in.

Sheet Thickness in.	F_u = 45 ksi						F_u = 65 ksi					
	Weld Length/Sheet Thickness, L / t Longitudinal					Transverse	Weld Length/Sheet Thickness, L / t Longitudinal					Transverse
	5	10	15	20	≥25		5	10	15	20	≥25	
	Ω						Ω					
	2.55	2.55	2.55	2.55	3.05	2.35	2.55	2.55	2.55	2.55	3.05	2.35
	ϕ						ϕ					
	0.60	0.60	0.60	0.60	0.50	0.65	0.60	0.60	0.60	0.60	0.50	0.65
0.036	1.54	1.46	1.38	1.30	1.22	1.62	2.22	2.11	1.99	1.87	1.76	2.34
0.048	2.05	1.94	1.84	1.73	1.62	2.16	2.96	2.81	2.65	2.50	2.34	3.12
0.060	2.57	2.43	2.30	2.16	2.03	2.70	3.71	3.51	3.32	3.12	2.93	3.90
0.075	3.21	3.04	2.87	2.70	2.53	3.38	4.63	4.39	4.14	3.90	3.66	4.88
0.090	3.85	3.65	3.44	3.24	3.04	4.05	5.56	5.27	4.97	4.68	4.39	5.85
0.105	4.49	4.25	4.02	3.78	3.54	4.73	6.48	6.14	5.80	5.46	5.12	6.83
0.135	5.77	5.47	5.16	4.86	4.56	6.08	8.34	7.90	7.46	7.02	6.58	8.78

Notes: (1) Available Strengths are:
ASD: $P'_n \times L / \Omega$
LRFD: $\phi P'_n \times L$
where L = weld length in inches
t = thickness of thinnest welded sheet
(2) Specification equation E2.4-4 must also be checked when t > 0.10 in.

Table IV - 2

Resistance ("Spot") Welds
Shear Strength

Ω = 2.35
ϕ = 0.65

Nominal Shear Strength, P_n, kips

Sheet Thickness (in.)								
0.012	0.024	0.036	0.048	0.060	0.090	0.105	0.135	0.165
0.216	0.599	1.09	1.66	2.30	4.18	5.24	7.58	9.09

Notes: Available Strengths are:
ASD: P_n / Ω
LRFD: ϕP_n

Table IV - 3

Arc Spot Welds
Shear of Sheet(s) Welded to a Thicker Supporting Member

Ω = 3.05 above double line
= 2.80 above heavy line
= 2.20 below heavy line

ϕ = 0.50 above double line
= 0.55 above heavy line
= 0.70 below heavy line

Nominal Shear Strength per Weld, P_n, kips

Total Sheet Thickness above Shear Plane, in.	F_u = 45 ksi			F_u = 65 ksi		
	Visible Diameter, in.			Visible Diameter, in.		
	1/2	5/8	3/4	1/2	5/8	3/4
0.015	0.497	0.576	0.695	0.662	0.833	1.00
0.020	0.842	0.874	0.920	1.04	1.10	1.33
0.025	1.18	1.32	1.36	1.57	1.63	1.68
0.030	1.40	1.77	1.90	2.02	2.28	2.34
0.035	1.61	2.04	2.48	2.33	2.95	3.11
0.040	1.82	2.32	2.81	2.63	3.35	3.98
0.045	2.03	2.58	3.14	2.93	3.73	4.54
0.050	2.23	2.85	3.47	3.22	4.11	5.01
0.055	2.42	3.10	3.78	3.50	4.48	5.47
0.060	2.61	3.36	4.10	3.78	4.85	5.92
0.065	2.80	3.60	4.41	4.04	5.21	6.37
0.070	2.98	3.85	4.71	4.30	5.56	6.81
0.075	3.16	4.08	5.01	4.56	5.90	7.24
0.080	3.33	4.32	5.31	4.80	6.23	7.66
0.085	3.49	4.54	5.60	5.04	6.56	8.08
0.090	3.65	4.77	5.88	5.28	6.89	8.49
0.095	3.81	4.98	6.16	5.50	7.20	8.90
0.100	3.96	5.20	6.44	5.72	7.51	9.30
0.105	4.11	5.41	6.70	5.93	7.81	9.68
0.110	4.25	5.61	6.97	6.13	8.10	10.1
0.115	4.38	5.81	7.23	6.33	8.39	10.4
0.120	4.51	6.00	7.48	6.52	8.67	10.8
0.125	4.64	6.19	7.73	6.70	8.94	11.2
0.130	4.76	6.37	7.98	6.88	9.20	11.5
0.135	4.88	6.55	8.22	7.05	9.46	11.9
0.140	4.99	6.72	8.45	7.21	9.71	12.2
0.145	5.10	6.89	8.68	7.36	9.95	12.5
0.150	5.20	7.05	8.91	7.51	10.2	12.9

Notes: (1) Available Strengths are:
ASD: P_n / Ω
LRFD: ϕP_n
(2) The nominal shear strength given in Eq. E2.2.1.2-1 of the Specification is not considered in Table IV-3 and must be checked.

Table IV - 4

Arc Spot Welds
Shear of Sheet Welded
to an Identical Sheet

Ω = 2.20
ϕ = 0.70

Nominal Shear Strength per Weld, P_n, kips

Total Sheet Thickness above Shear Plane, in.	F_u = 45 ksi		
	Visible Diameter, in.		
	1/2	5/8	3/4
0.030	1.05	1.33	1.60
0.035	1.21	1.53	1.86
0.040	1.37	1.74	2.11
0.045	1.52	1.94	2.36
0.050	1.67	2.13	2.60
0.055	1.82	2.33	2.84
0.060	1.96	2.52	3.07

Notes: (1) Available Strengths are:
ASD: P_n / Ω
LRFD: ϕP_n
(2) F_{xx} > 45 ksi required.

Table IV - 5

Arc Spot Welds
Tension

Ω = 2.50 for panel and deck
= 3.00 for other applications

ϕ = 0.60 for panel and deck
= 0.50 for other applications

Nominal Shear Strength per Weld, P_n, kips

Total Sheet Thickness, in.	F_y = 33 ksi, F_u = 45 ksi			F_y = 50 ksi, F_u = 65 ksi		
	Visible Diameter, in.			Visible Diameter, in.		
	1/2	5/8	3/4	1/2	5/8	3/4
0.015	0.487	0.613	0.738	0.639	0.804	0.969
0.020	0.643	0.810	0.977	0.844	1.06	1.28
0.025	0.795	1.00	1.21	1.04	1.32	1.59
0.030	0.944	1.19	1.45	1.24	1.57	1.90
0.035	1.09	1.38	1.68	1.43	1.81	2.20
0.040	1.23	1.57	1.90	1.62	2.06	2.50
0.045	1.37	1.75	2.12	1.80	2.29	2.79
0.050	1.51	1.92	2.34	1.98	2.53	3.08
0.055	1.64	2.10	2.56	2.15	2.76	3.36
0.060	1.77	2.27	2.77	2.32	2.98	3.64
0.065	1.89	2.44	2.98	2.48	3.20	3.91
0.070	2.01	2.60	3.19	2.65	3.41	-
0.075	2.13	2.76	3.39	2.80	3.63	-
0.080	2.25	2.92	3.59	2.95	3.83	-
0.085	2.36	3.07	3.78	3.10	4.03	-
0.090	2.47	3.22	3.98	3.24	-	-
0.095	2.58	3.37	4.17	3.38	-	-
0.100	2.68	3.51	4.35	3.52	-	-
0.105	2.78	3.66	-	3.64	-	-
0.110	2.87	3.79	-	3.77	-	-
0.115	2.96	3.93	-	3.89	-	-
0.120	3.05	4.06	-	4.01	-	-
0.125	3.14	4.18	-	-	-	-
0.130	3.22	4.31	-	-	-	-
0.135	3.30	4.43	-	-	-	-
0.140	3.37	-	-	-	-	-
0.145	3.45	-	-	-	-	-
0.150	3.51	-	-	-	-	-

Notes:
(1) Available Strengths are:
ASD: P_n / Ω
LRFD: ϕP_n
(2) The nominal tensile strength given in Eq. E2.2.2-1 of the Specification is not considered in Table IV-5 and must be checked.
(3) The limitations related to weld electrode strength, F_{xx}, have not been checked in this table and must be checked per Section E2.2.2..
(4) Dashed values indicate that the limit $td_aF_u \leq 3$ kips has not been satisfied.

SECTION 2 - BOLTS

Bolts, washers and nuts approved for use with cold-formed members are listed in the *Specification* in Section E3. The application must comply with the requirements set forth in Section E3. The *Specification* applies to the bolting of cold-formed steel structural members in which the thickness of the thinnest connected part is less than 3/16 inch. For connections where the thinnest connection part equals 3/16 inch or thicker refer to the AISC specification[1]. The area resisting failure due to shear or tension is determined by deducting the bolt hole size along the corresponding failure surface. A standard hole is defined for bolts less than 1/2 inch in diameter as the diameter of the bolt plus 1/32 in. For bolts equal to or greater than 1/2 in., the standard hole size is taken as the bolt diameter plus 1/16 inch. Requirements for bolted slip critical connections are not contained in the *Specification*.

Bolted connections are subject to the limit states of

1. Shear governed by spacing and edge distance
2. Rupture in net section (shear lag)
3. Bearing on the base material
4. Bolt strength

Spacing and Edge Distance: Bolt spacing and edge distance provisions are found in Section E3.1 in Appendix A (or B for Canada). The available strengths are based on shearing of connected materials between the outside bolt and the edge of the material or shearing of the connected material between bolt holes.

Rupture in net section: Rupture of the net cross-section subject to tension must be evaluated using Section E3.2 in Appendix A (or B for Canada). This section includes provisions for connections with staggered and non-staggered hole patterns as well as members in which forces are transferred by less than all of the cross-section elements.

Bearing: Section E3.3 in the main body of the *Specification* provides strength checks based on the bearing strength of the connected material. Separate checks are provided for the cases where bolt hole deformation is, and is not, considered.

Bolt Strength: The strength of bolts is evaluated using the provision of Section E3.4 in Appendix A (or Appendix B for Canada). Strengths are provided for shear, tension and the interaction of shear and tension.

2.1 Notes On The Tables

Shown in Tables IV-6 and IV-7 are tabulated values for the nominal tension strength and the nominal shear strength for A307, A325, A449, A354 and A490 bolts. Available strengths can be found directly from the table for ASD by dividing by Ω, and for LRFD by multiplying by ϕ.

Provided in Tables IV-8a, IV-8b and IV-8c are bearing strengths under various shear conditions for steels with tensile strengths of 45 ksi and 65 ksi. The allowable strength for ASD can be found by dividing by Ω, and for LRFD by multiplying by ϕ.

2.2 Bolted Connection Design Tables

[1] AISC-360, *Specification for Structural Steel Buildings*, American Institute of Steel Construction, Chicago, IL, 2005

Table IV - 6

Bolts
Tension

Ω = See Table
ϕ = 0.75

Nominal Tension Strength, P_n, kips

ASTM Designation	F_y ksi	F_u ksi	Diameter in.	Ω	F_{nt} ksi	Nominal Bolt Diameter, in.							
						1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4
						Gross Area, in.^{2}							
						.0491	.0767	.1104	.1503	.1963	.2485	.3068	.4418
A307	-	60	< 1/2	2.25	40.5	1.99	3.11	4.47	6.09				
			≥ 1/2	2.25	45.0					8.84	11.2	13.8	19.9
A325	92	120	≥ 1/2	2.0	90.0					17.7	22.4	27.6	39.8
A449	92	120	< 1/2	2.0	81.0	3.98	6.21	8.95	12.2				
A354 Gr. BD	130	150	< 1/2	2.0	101.0	4.96	7.75	11.2	15.2				
A490	-	150	≥ 1/2	2.0	112.5					22.1	28.0	34.5	49.7

Note: Available Strengths are:
ASD: P_n / Ω
LRFD: ϕP_n

Table IV - 7

Bolts
Shear

Ω = 2.4
ϕ = 0.65

Nominal Shear Strength, P_n, kips

ASTM Designation	Type (2)	Diameter in.	F_{nv} ksi	Nominal Bolt Diameter, in.							
				1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4
				Gross Area, in.^{2}							
				.0491	.0767	.1104	.1503	.1963	.2485	.3068	.4418
A307	N or X	<1/2	24.0	1.18	1.84	2.65	3.61				
		≥ 1/2	27.0					5.30	6.71	8.28	11.9
A325	N	≥ 1/2	54.0					10.6	13.4	16.6	23.9
	X		72.0					14.1	17.9	22.1	31.8
A449	N	< 1/2	47.0	2.31	3.60	5.19	7.07				
	X		72.0	3.53	5.52	7.95	10.8				
A354 Gr. BD	N	<1/2	59.0	2.90	4.53	6.52	8.87				
	X		90.0	4.42	6.90	9.94	13.5				
A490	N	≥ 1/2	67.5					13.3	16.8	20.7	29.8
	X		90.0					17.7	22.4	27.6	39.8

Notes: (1) Available Strengths are:
ASD: P_n / Ω
LRFD: ϕP_n
(2) Type N has threads included in a shear plane
Type X has threads excluded from all shear planes

Table IV - 8a

Bolts
Bearing on Connected Members
Inside Sheet of Double Shear Connections
Bolt Hole Deformation Not Considered

Ω = 2.50
ϕ = 0.60

Nominal Bearing Strength, P_n, kips

t (in.)	F_u = 45 ksi								F_u = 65 ksi							
	Nominal Bolt Diameter, in.								Nominal Bolt Diameter, in.							
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4	1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4
0.024	1.06	1.21	1.31	1.37	1.38	1.45	1.62	1.94	1.53	1.75	1.90	1.98	1.99	2.10	2.33	2.80
0.036	1.62	2.02	2.39	2.62	2.81	2.95	3.05	3.10	2.33	2.92	3.45	3.79	4.06	4.27	4.40	4.47
0.048	2.15	2.69	3.23	3.77	4.25	4.57	4.84	5.25	3.11	3.89	4.67	5.45	6.14	6.60	7.00	7.59
0.060	2.69	3.37	4.04	4.71	5.39	6.06	6.64	7.41	3.89	4.86	5.84	6.81	7.78	8.75	9.59	10.7
0.075	3.37	4.21	5.05	5.89	6.73	7.57	8.42	10.10	4.86	6.08	7.29	8.51	9.73	10.9	12.2	14.6
0.090	4.04	5.05	6.06	7.07	8.08	9.09	10.1	12.1	5.84	7.29	8.75	10.2	11.7	13.1	14.6	17.5
0.105	4.71	5.89	7.07	8.25	9.43	10.6	11.8	14.1	6.81	8.51	10.2	11.9	13.6	15.3	17.0	20.4
0.135	6.06	7.57	9.09	10.6	12.1	13.6	15.1	18.2	8.75	10.9	13.1	15.3	17.5	19.7	21.9	26.3
0.165	7.41	9.26	11.1	13.0	14.8	16.7	18.5	22.2	10.70	13.4	16.0	18.7	21.4	24.1	26.7	32.1

Notes: Available Strengths are:
ASD: P_n / Ω
LRFD: ϕP_n

Table IV - 8b

Bolts
Bearing on Connected Members
Outside Sheets of Connections With Washers on Both Sides
Bolt Hole Deformation Not Considered

Ω = 2.50
ϕ = 0.60

Nominal Bearing Strength, P_n, kips

t (in.)	F_u = 45 ksi								F_u = 65 ksi							
	Nominal Bolt Diameter, in.								Nominal Bolt Diameter, in.							
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4	1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4
0.024	0.799	0.911	0.987	1.03	1.03	1.09	1.22	1.46	1.15	1.32	1.43	1.49	1.50	1.58	1.75	2.11
0.036	1.22	1.52	1.80	1.97	2.12	2.22	2.29	2.33	1.75	2.19	2.60	2.85	3.06	3.21	3.31	3.36
0.048	1.62	2.03	2.43	2.84	3.19	3.44	3.64	3.95	2.34	2.92	3.51	4.09	4.61	4.96	5.26	5.70
0.060	2.03	2.53	3.04	3.54	4.05	4.56	4.99	5.57	2.92	3.66	4.39	5.12	5.85	6.58	7.21	8.04
0.075	2.53	3.16	3.80	4.43	5.06	5.70	6.33	7.59	3.66	4.57	5.48	6.40	7.31	8.23	9.14	11.0
0.090	3.04	3.80	4.56	5.32	6.07	6.83	7.59	9.11	4.39	5.48	6.58	7.68	8.77	9.87	11.0	13.2
0.105	3.54	4.43	5.32	6.20	7.09	7.97	8.86	10.6	5.12	6.40	7.68	8.96	10.2	11.5	12.8	15.4
0.135	4.56	5.70	6.83	7.97	9.11	10.3	11.4	13.7	6.58	8.23	9.87	11.5	13.2	14.8	16.5	19.7
0.165	5.57	6.96	8.35	9.75	11.1	12.5	13.9	16.7	8.04	10.1	12.1	14.1	16.1	18.1	20.1	24.1

Notes: Available Strengths are:
ASD: P_n / Ω
LRFD: ϕP_n

Table IV - 8c

Bolts
Bearing on Connected Members
Outside Sheets of Connections Without Washers on Both Sides
Bolt Hole Deformation Not Considered

Ω = 2.50
ϕ = 0.60

Nominal Bearing Strength, P_n, kips

t (in.)	F_u = 45 ksi								F_u = 65 ksi							
	Nominal Bolt Diameter, in.								Nominal Bolt Diameter, in.							
	1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4	1/4	5/16	3/8	7/16	1/2	9/16	5/8	3/4
0.024	0.599	0.683	0.740	0.772	0.776	0.820	0.911	1.09	0.865	0.986	1.07	1.11	1.12	1.18	1.32	1.58
0.036	0.911	1.14	1.35	1.48	1.59	1.67	1.72	1.75	1.32	1.65	1.95	2.14	2.29	2.41	2.48	2.52
0.048	1.22	1.52	1.82	2.13	2.40	2.58	2.73	2.96	1.75	2.19	2.63	3.07	3.46	3.72	3.95	4.28
0.060	1.52	1.90	2.28	2.66	3.04	3.42	3.74	4.18	2.19	2.74	3.29	3.84	4.39	4.94	5.41	6.03
0.075	1.90	2.37	2.85	3.32	3.80	4.27	4.75	5.70	2.74	3.43	4.11	4.80	5.48	6.17	6.86	8.23
0.090	2.28	2.85	3.42	3.99	4.56	5.13	5.70	6.83	3.29	4.11	4.94	5.76	6.58	7.40	8.23	9.87
0.105	2.66	3.32	3.99	4.65	5.32	5.98	6.64	7.97	3.84	4.80	5.76	6.72	7.68	8.64	9.60	11.5
0.135	3.42	4.27	5.13	5.98	6.83	7.69	8.54	10.3	4.94	6.17	7.40	8.64	9.87	11.1	12.3	14.8
0.165	4.18	5.22	6.26	7.31	8.35	9.40	10.4	12.5	6.03	7.54	9.05	10.6	12.1	13.6	15.1	18.1

Note: **Available Strengths are:**
ASD: P_{ns} / Ω
LRFD: ϕP_{ns}

SECTION 3 - SCREWS

Requirements for screw connections are listed in the *Specification* in Section E4. Application is limited to self-tapping screws with nominal screw diameters greater than 0.08 in. and less than or equal to 0.25 in. The screws must be thread forming or thread cutting, with or without a self-drilling point.

Screwed connections in shear are subject to the limit states of:

1. Tilting and Bearing
2. End distance
3. Shear in screws

Tilting and Bearing: The strength of screws in shear can be limited by simple bearing on the connected material, or by more complex modes involving the tilting of the screw and subsequent pullout. Section E4.3.1 provides strength checks for these limit states.

End Distance: Strength calculations based on edge distance, analogous to those for bolted connections, are found in Section E4.3.2 located in Appendix A (or Appendix B for Canada)

Shear in Screws: Connection strength is also limited by the shear strength of the screws themselves. Section E4.3.3 requires that the nominal screw strength be provided by the screw manufacturer or an independent testing laboratory. The *Specification* provides resistance and safety factors, but permits the use of testing to calculate more favorable factors.

Screwed connections in tension are subject to the limit states of:

1. Pull-out
2. Pull-over
3. Tension in screws

Pull-out: Screws strengths for the limit state of the screw threads pulling out of the connecting material is calculated using Section E4.4.1. The strength is limited by the thickness and material strength of the material into which the screw is anchored.

Pull-over: Screws strengths for the limit state of the top sheet of material pulling over the screw head and/or washer is calculated using Section E4.4.2. The strength is limited by the thickness and material strength of the material directly under the screw head, as well as the type of screw head.

Tension in Screws: Connection strength is also limited by the tensile strength of the screws themselves. Section E4.4.3 requires that the nominal screw strength be provided by the screw manufacturer or an independent testing laboratory. The *Specification* provides resistance and safety factors, but permits the use of testing to calculate more favorable factors.

3.1 Notes On The Tables

Provided in Tables IV-9a through IV-9d are the nominal shear strengths of screwed connections with designations from #6 to 1/4 inch, which connect various sheet thickness combinations. These are presented for sheets with tensile strengths of 45 ksi and 65 ksi.

Provided in Tables IV-10a through IV-10d are the nominal pullout strengths of screwed connections with designations from #6 to 1/4 inch, in various thicknesses of material. These are presented for sheets with tensile strengths of 45 ksi and 65 ksi.

Provided in Tables IV-11a through IV-11d are the nominal pullover strengths of connections with hex head and hex washer head screws with designations from #6 to 1/4 inch, in various thicknesses of material. These are presented for sheets with tensile strengths of 45 ksi and 65 ksi. ANSI/ASME standard screw head diameters are used in the calculations and are listed in the tables. Larger or smaller diameters will results in different strengths. The hex washer head screw values are also applicable to other screws with washers of the listed diameter having a minimum thickness of 0.050 in.

The nominal strengths can be determined by interpolating within the Tables. The allowable strength for ASD can be found by dividing the nominal strength by Ω. The design strength for LRFD can be found by multiplying the nominal strength by ϕ.

Tables are provided for both the thicknesses used in the representative sections and design thicknesses used by SSMA. The former set of tables are more convenient for interpolation. The latter set of tables provide direct solutions for SSMA studs and tracks without the need for interpolation.

Note that shear and tensile strengths of the fasteners must be determined by the manufacturer through tests.

3.2 Screwed Connection Design Tables

Table IV - 9a

Screws
Shear of Sheet - F_u = 45 ksi
Representative Thicknesses

Ω = 3.0
ϕ = 0.5

Nominal Shear Strength, P_{ns}, kips

Screw Designation	Diameter in.	Thickness of member in contact with screw head, in.	Thickness of member not in contact with the screw head, in.						
			0.036	0.048	0.060	0.075	0.090	0.105	0.135
#6	0.138	0.036	0.480	0.604	0.604	0.604	0.604	0.604	0.604
		0.048	0.480	0.738	0.805	0.805	0.805	0.805	0.805
		0.060	0.480	0.738	1.01	1.01	1.01	1.01	1.01
		0.075	0.480	0.738	1.01	1.26	1.26	1.26	1.26
		0.090	0.480	0.738	1.01	1.26	1.51	1.51	1.51
		0.105	0.480	0.738	1.01	1.26	1.51	1.76	1.76
		0.135	0.480	0.738	1.01	1.26	1.51	1.76	2.26
#8	0.164	0.036	0.523	0.717	0.717	0.717	0.717	0.717	0.717
		0.048	0.523	0.805	0.956	0.956	0.956	0.956	0.956
		0.060	0.523	0.805	1.12	1.20	1.20	1.20	1.20
		0.075	0.523	0.805	1.12	1.49	1.49	1.49	1.49
		0.090	0.523	0.805	1.12	1.49	1.79	1.79	1.79
		0.105	0.523	0.805	1.12	1.49	1.79	2.09	2.09
		0.135	0.523	0.805	1.12	1.49	1.79	2.09	2.69
#10	0.190	0.036	0.563	0.831	0.831	0.831	0.831	0.831	0.831
		0.048	0.563	0.866	1.11	1.11	1.11	1.11	1.11
		0.060	0.563	0.866	1.21	1.39	1.39	1.39	1.39
		0.075	0.563	0.866	1.21	1.69	1.73	1.73	1.73
		0.090	0.563	0.866	1.21	1.69	2.08	2.08	2.08
		0.105	0.563	0.866	1.21	1.69	2.08	2.42	2.42
		0.135	0.563	0.866	1.21	1.69	2.08	2.42	3.12
#12	0.216	0.036	0.600	0.928	0.945	0.945	0.945	0.945	0.945
		0.048	0.600	0.924	1.26	1.26	1.26	1.26	1.26
		0.060	0.600	0.924	1.29	1.57	1.57	1.57	1.57
		0.075	0.600	0.924	1.29	1.80	1.97	1.97	1.97
		0.090	0.600	0.924	1.29	1.80	2.36	2.36	2.36
		0.105	0.600	0.924	1.29	1.80	2.36	2.76	2.76
		0.135	0.600	0.924	1.29	1.80	2.36	2.76	3.54
1/4 in.	0.250	0.036	0.645	1.02	1.09	1.09	1.09	1.09	1.09
		0.048	0.645	0.994	1.40	1.46	1.46	1.46	1.46
		0.060	0.645	0.994	1.39	1.82	1.82	1.82	1.82
		0.075	0.645	0.994	1.39	1.94	2.28	2.28	2.28
		0.090	0.645	0.994	1.39	1.94	2.55	2.73	2.73
		0.105	0.645	0.994	1.39	1.94	2.55	3.19	3.19
		0.135	0.645	0.994	1.39	1.94	2.55	3.19	4.10

Note: Available Strengths are:
ASD: P_{ns} / Ω
LRFD: ϕP_{ns}

Table IV - 9b

Screws
Shear of Sheet - F_u = 65 ksi
Representative Thicknesses

Ω = 3.0
ϕ = 0.5

Nominal Shear Strength, P_{ns}, kips

Screw Designation	Diameter in.	Thickness of member in contact with screw head, in.	Thickness of member not in contact with screw head, in.						
			0.036	0.048	0.060	0.075	0.090	0.105	0.135
#6	0.138	0.036	0.693	0.872	0.872	0.872	0.872	0.872	0.872
		0.048	0.693	1.07	1.16	1.16	1.16	1.16	1.16
		0.060	0.693	1.07	1.45	1.45	1.45	1.45	1.45
		0.075	0.693	1.07	1.45	1.82	1.82	1.82	1.82
		0.090	0.693	1.07	1.45	1.82	2.18	2.18	2.18
		0.105	0.693	1.07	1.45	1.82	2.18	2.54	2.54
		0.135	0.693	1.07	1.45	1.82	2.18	2.54	3.27
#8	0.164	0.036	0.755	1.04	1.04	1.04	1.04	1.04	1.04
		0.048	0.755	1.16	1.38	1.38	1.38	1.38	1.38
		0.060	0.755	1.16	1.62	1.73	1.73	1.73	1.73
		0.075	0.755	1.16	1.62	2.16	2.16	2.16	2.16
		0.090	0.755	1.16	1.62	2.16	2.59	2.59	2.59
		0.105	0.755	1.16	1.62	2.16	2.59	3.02	3.02
		0.135	0.755	1.16	1.62	2.16	2.59	3.02	3.89
#10	0.190	0.036	0.813	1.20	1.20	1.20	1.20	1.20	1.20
		0.048	0.813	1.25	1.60	1.60	1.60	1.60	1.60
		0.060	0.813	1.25	1.75	2.00	2.00	2.00	2.00
		0.075	0.813	1.25	1.75	2.44	2.50	2.50	2.50
		0.090	0.813	1.25	1.75	2.44	3.00	3.00	3.00
		0.105	0.813	1.25	1.75	2.44	3.00	3.50	3.50
		0.135	0.813	1.25	1.75	2.44	3.00	3.50	4.50
#12	0.216	0.036	0.867	1.34	1.36	1.36	1.36	1.36	1.36
		0.048	0.867	1.33	1.82	1.82	1.82	1.82	1.82
		0.060	0.867	1.33	1.86	2.27	2.27	2.27	2.27
		0.075	0.867	1.33	1.86	2.61	2.84	2.84	2.84
		0.090	0.867	1.33	1.86	2.61	3.41	3.41	3.41
		0.105	0.867	1.33	1.86	2.61	3.41	3.98	3.98
		0.135	0.867	1.33	1.86	2.61	3.41	3.98	5.12
1/4 in.	0.250	0.036	0.932	1.47	1.58	1.58	1.58	1.58	1.58
		0.048	0.932	1.44	2.02	2.11	2.11	2.11	2.11
		0.060	0.932	1.44	2.01	2.63	2.63	2.63	2.63
		0.075	0.932	1.44	2.01	2.80	3.29	3.29	3.29
		0.090	0.932	1.44	2.01	2.80	3.69	3.95	3.95
		0.105	0.932	1.44	2.01	2.80	3.69	4.61	4.61
		0.135	0.932	1.44	2.01	2.80	3.69	4.61	5.92

Note: **Available Strengths are:**
ASD: P_{ns} / Ω
LRFD: ϕP_{ns}

Table IV – 9c

Screws
Shear of Sheet - F_u = 45 ksi
SSMA Design Thicknesses

Ω = 3.0
ϕ = 0.5

Nominal Shear Strength, P_{ns}, kips

Screw Designation	Diameter in.	Thickness of member in contact with screw head, in.	Thickness of member not in contact with the screw head, in.							
			0.0188	0.0283	0.0312	0.0346	0.0451	0.0566	0.0713	0.1017
#6	0.138	0.0188	0.181	0.315	0.315	0.315	0.315	0.315	0.315	0.315
		0.0283	0.181	0.334	0.393	0.455	0.475	0.475	0.475	0.475
		0.0312	0.181	0.334	0.387	0.457	0.523	0.523	0.523	0.523
		0.0346	0.181	0.334	0.387	0.452	0.580	0.580	0.580	0.580
		0.0451	0.181	0.334	0.387	0.452	0.672	0.756	0.756	0.756
		0.0566	0.181	0.334	0.387	0.452	0.672	0.945	0.949	0.949
		0.0713	0.181	0.334	0.387	0.452	0.672	0.945	1.20	1.20
		0.1017	0.181	0.334	0.387	0.452	0.672	0.945	1.20	1.71
#8	0.164	0.0188	0.197	0.368	0.375	0.375	0.375	0.375	0.375	0.375
		0.0283	0.197	0.364	0.432	0.503	0.564	0.564	0.564	0.564
		0.0312	0.197	0.364	0.422	0.502	0.622	0.622	0.622	0.622
		0.0346	0.197	0.364	0.422	0.493	0.689	0.689	0.689	0.689
		0.0451	0.197	0.364	0.422	0.493	0.733	0.899	0.899	0.899
		0.0566	0.197	0.364	0.422	0.493	0.733	1.03	1.13	1.13
		0.0713	0.197	0.364	0.422	0.493	0.733	1.03	1.42	1.42
		0.1017	0.197	0.364	0.422	0.493	0.733	1.03	1.42	2.03
#10	0.190	0.0188	0.212	0.406	0.434	0.434	0.434	0.434	0.434	0.434
		0.0283	0.212	0.392	0.468	0.548	0.653	0.653	0.653	0.653
		0.0312	0.212	0.392	0.454	0.544	0.720	0.720	0.720	0.720
		0.0346	0.212	0.392	0.454	0.530	0.791	0.799	0.799	0.799
		0.0451	0.212	0.392	0.454	0.530	0.789	1.04	1.04	1.04
		0.0566	0.212	0.392	0.454	0.530	0.789	1.11	1.31	1.31
		0.0713	0.212	0.392	0.454	0.530	0.789	1.11	1.57	1.65
		0.1017	0.212	0.392	0.454	0.530	0.789	1.11	1.57	2.35
#12	0.216	0.0188	0.226	0.444	0.488	0.493	0.493	0.493	0.493	0.493
		0.0283	0.226	0.418	0.502	0.592	0.743	0.743	0.743	0.743
		0.0312	0.226	0.418	0.484	0.584	0.819	0.819	0.819	0.819
		0.0346	0.226	0.418	0.484	0.565	0.855	0.908	0.908	0.908
		0.0451	0.226	0.418	0.484	0.565	0.841	1.18	1.18	1.18
		0.0566	0.226	0.418	0.484	0.565	0.841	1.18	1.49	1.49
		0.0713	0.226	0.418	0.484	0.565	0.841	1.18	1.67	1.87
		0.1017	0.226	0.418	0.484	0.565	0.841	1.18	1.67	2.67
1/4 in.	0.250	0.0188	0.244	0.491	0.543	0.571	0.571	0.571	0.571	0.571
		0.0283	0.244	0.450	0.544	0.646	0.860	0.860	0.860	0.860
		0.0312	0.244	0.450	0.521	0.633	0.918	0.948	0.948	0.948
		0.0346	0.244	0.450	0.521	0.608	0.935	1.05	1.05	1.05
		0.0451	0.244	0.450	0.521	0.608	0.905	1.29	1.37	1.37
		0.0566	0.244	0.450	0.521	0.608	0.905	1.27	1.72	1.72
		0.0713	0.244	0.450	0.521	0.608	0.905	1.27	1.80	2.17
		0.1017	0.244	0.450	0.521	0.608	0.905	1.27	1.80	3.06

Note: Available Strengths are:
ASD: P_{ns} / Ω
LRFD: ϕP_{ns}

Table IV – 9d

Screws
Shear of Sheet - F_u = 65 ksi
SSMA Design Thicknesses

Ω = 3.0
ϕ = 0.5

Nominal Shear Strength, P_{ns}, kips

Screw Designation	Diameter in.	Thickness of member in contact with screw head, in.	Thickness of member not in contact with screw head, in.		
			0.0566	0.0713	0.1017
#6	0.138	0.0566	1.37	1.37	1.37
		0.0713	1.37	1.73	1.73
		0.1017	1.37	1.73	2.46
#8	0.164	0.0566	1.49	1.63	1.63
		0.0713	1.49	2.05	2.05
		0.1017	1.49	2.05	2.93
#10	0.190	0.0566	1.60	1.89	1.89
		0.0713	1.60	2.27	2.38
		0.1017	1.60	2.27	3.39
#12	0.216	0.0566	1.71	2.15	2.15
		0.0713	1.71	2.42	2.70
		0.1017	1.71	2.42	3.86
1/4 in.	0.250	0.0566	1.84	2.48	2.48
		0.0713	1.84	2.60	3.13
		0.1017	1.84	2.60	4.43

Note: **Available Strengths are:**
ASD: P_{ns} / Ω
LRFD: ϕP_{ns}

Table IV - 10a

Screws
Pull-Out - F_u = 45 ksi
Representative Thicknesses

Ω = 3.0
ϕ = 0.5

Nominal Pullout Strength, P_{not}, kips

Screw Designation	Diameter in.	Thickness of member not in contact with the screw head, in.						
		0.036	0.048	0.060	0.075	0.090	0.105	0.135
#6	0.138	0.190	0.253	0.317	0.396	0.475	0.554	0.713
#8	0.164	0.226	0.301	0.376	0.470	0.565	0.659	0.847
#10	0.190	0.262	0.349	0.436	0.545	0.654	0.763	0.981
#12	0.216	0.297	0.397	0.496	0.620	0.744	0.868	1.12
1/4 in.	0.250	0.344	0.459	0.574	0.717	0.861	1.00	1.29

Table IV - 10b

Screws
Pull-Out - F_u = 65 ksi
Representative Thicknesses

Ω = 3.0
ϕ = 0.5

Nominal Pullout Strength, P_{not}, kips

Screw Designation	Diameter in.	Thickness of member not in contact with the screw head, in.						
		0.036	0.048	0.060	0.075	0.090	0.105	0.135
#6	0.138	0.274	0.366	0.457	0.572	0.686	0.801	1.03
#8	0.164	0.326	0.435	0.544	0.680	0.815	0.951	1.22
#10	0.190	0.378	0.504	0.630	0.787	0.945	1.10	1.42
#12	0.216	0.430	0.573	0.716	0.895	1.07	1.25	1.61
1/4 in.	0.250	0.497	0.663	0.829	1.04	1.24	1.45	1.86

Note: **Available Strengths are:**
ASD: P_{not} / Ω
LRFD: ϕP_{not}

Table IV – 10c

Screws
Pull-Out - F_u = 45 ksi
SSMA Design Thicknesses

Ω = 3.0
ϕ = 0.5

Nominal Pullout Strength, P_{not}, kips

Screw Designation	Diameter in.	Thickness of member not in contact with the screw head, in.							
		0.0188	0.0283	0.0312	0.0346	0.0451	0.0566	0.0713	0.1017
#6	0.138	0.099	0.149	0.165	0.183	0.238	0.299	0.376	0.537
#8	0.164	0.118	0.178	0.196	0.217	0.283	0.355	0.447	0.638
#10	0.190	0.137	0.206	0.227	0.251	0.328	0.411	0.518	0.739
#12	0.216	0.155	0.234	0.258	0.286	0.373	0.468	0.589	0.840
1/4 in.	0.250	0.180	0.271	0.298	0.331	0.431	0.541	0.682	0.973

Table IV – 10d

Screws
Pull-Out - F_u = 65 ksi
SSMA Design Thicknesses

Ω = 3.0
ϕ = 0.5

Nominal Pullout Strength, P_{not}, kips

Screw Designation	Diameter in.	Thickness of member not in contact with the screw head, in.		
		0.0566	0.0713	0.1017
#6	0.138	0.432	0.544	0.775
#8	0.164	0.513	0.646	0.922
#10	0.190	0.594	0.748	1.07
#12	0.216	0.675	0.851	1.21
1/4 in.	0.250	0.782	0.985	1.40

Note: **Available Strengths are:**
ASD: P_{not} / Ω
LRFD: ϕP_{not}

Table IV - 11a

Hex Head Screws
Pull-Over - F_u = 45 ksi
Representative Thicknesses

$\Omega = 3.0$
$\phi = 0.5$

Nominal Pullover Strength, P_{nov}, kips

Hex Head Screws without Washers								
Screw Designation	Hex Head Diameter in.	Thickness of member in contact with the screw head, in.						
		0.036	0.048	0.060	0.075	0.090	0.105	0.135
#6	0.272	0.661	0.881	1.10	1.38	1.65	1.93	2.48
#8	0.272	0.661	0.881	1.10	1.38	2.65	1.93	2.48
#10	0.340	0.826	1.10	1.38	1.72	2.07	2.41	3.10
#12	0.340	0.826	1.10	1.38	1.72	2.07	2.41	3.10
1/4 in.	0.409	0.994	1.33	1.66	2.07	2.48	2.90	3.73
Hex Washer Head Screws								
Screw Designation	Washer Head Diameter in.	Thickness of member in contact with the washer head, in.						
		0.036	0.048	0.060	0.075	0.090	0.105	0.135
#6	0.315	0.765	1.02	1.28	1.59	1.91	2.23	2.87
#8	0.335	0.814	1.09	1.36	1.70	2.04	2.37	3.05
#10	0.399	0.970	1.29	1.62	2.02	2.42	2.83	3.64
#12	0.415	1.01	1.34	1.68	2.10	2.52	2.94	3.78
1/4 in.	0.500	1.22	1.62	2.03	2.53	3.04	3.54	4.56

Table IV - 11b

Hex Head Screws
Pull-Over - F_u = 65 ksi
Representative Thicknesses

$\Omega = 3.0$
$\phi = 0.5$

Nominal Pullover Strength, P_{nov}, kips

Hex Head Screws without Washers								
Screw Designation	Hex Head Diameter in.	Thickness of member in contact with the washer head, in.						
		0.036	0.048	0.060	0.075	0.090	0.105	0.135
#6	0.272	0.955	1.27	1.59	1.99	2.39	2.78	3.58
#8	0.272	0.955	1.27	1.59	1.99	2.39	2.78	3.58
#10	0.340	1.19	1.59	1.99	2.49	2.98	3.48	4.48
#12	0.340	1.19	1.59	1.99	2.49	2.98	3.48	4.48
1/4 in.	0.409	1.44	1.91	2.39	2.99	3.59	4.19	5.38
Hex Washer Head Screws								
Screw Designation	Washer Head Diameter in.	Thickness of member in contact with the washer, in.						
		0.036	0.048	0.060	0.075	0.090	0.105	0.135
#6	0.315	1.11	1.47	1.84	2.30	2.76	3.22	4.15
#8	0.335	1.18	1.57	1.96	2.45	2.94	3.43	4.41
#10	0.399	1.40	1.87	2.33	2.92	3.50	4.08	5.25
#12	0.415	1.46	1.94	2.43	3.03	3.64	4.25	5.46
1/4 in.	0.500	1.76	2.34	2.93	3.66	4.39	5.12	6.58

Notes: Available Strengths are:
ASD: P_{nov} / Ω
LRFD: ϕP_{nov}

Table IV – 11c

Hex Head Screws
Pull-Over - F_u = 45 ksi
SSMA Design Thicknesses

Ω = 3.0
ϕ = 0.5

Nominal Pullover Strength, P_{nov}, kips

Hex Head Screws without Washers									
Screw Designation	Hex Head Diameter in.	Thickness of member in contact with the screw head, in.							
		0.0188	0.0283	0.0312	0.0346	0.0451	0.0566	0.0713	0.1017
#6	0.272	0.345	0.520	0.573	0.635	0.828	1.04	1.31	1.87
#8	0.272	0.345	0.520	0.573	0.635	0.828	1.04	1.31	1.87
#10	0.340	0.431	0.649	0.716	0.794	1.04	1.30	1.64	2.33
#12	0.340	0.431	0.649	0.716	0.794	1.04	1.30	1.64	2.33
1/4 in.	0.409	0.519	0.781	0.861	0.955	1.25	1.56	1.97	2.81
Hex Washer Head Screws									
Screw Designation	Washer Head Diameter in.	Thickness of member in contact with the washer head, in.							
		0.0188	0.0283	0.0312	0.0346	0.0451	0.0566	0.0713	0.1017
#6	0.315	0.400	0.602	0.663	0.736	0.959	1.20	1.52	2.16
#8	0.335	0.425	0.640	0.706	0.782	1.02	1.28	1.61	2.30
#10	0.399	0.506	0.762	0.840	0.932	1.21	1.52	1.92	2.74
#12	0.415	0.527	0.793	0.874	0.969	1.26	1.59	2.00	2.85
1/4 in.	0.500	0.635	0.955	1.05	1.17	1.52	1.91	2.41	3.43

Table IV – 11d

Hex Head Screws
Pull-Over - F_u = 65 ksi
SSMA Design Thicknesses

Ω = 3.0
ϕ = 0.5

Nominal Pullover Strength, P_{nov}, kips

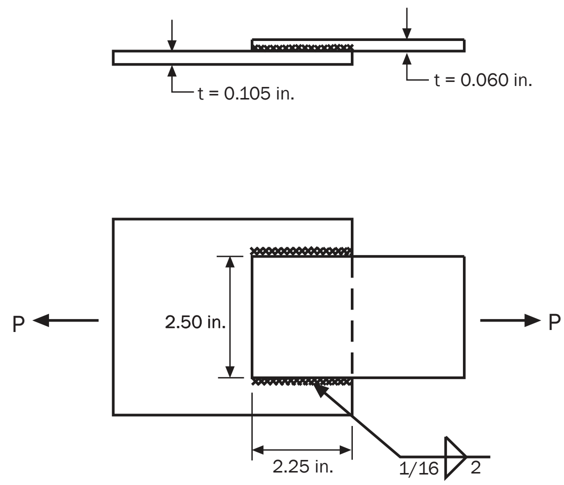
Hex Head Screws without Washers				
Screw Designation	Hex Head Diameter in.	Thickness of member in contact with the washer head, in.		
		0.0566	0.0713	0.1017
#6	0.272	1.50	1.89	2.70
#8	0.272	1.50	1.89	2.70
#10	0.340	1.88	2.36	3.37
#12	0.340	1.88	2.36	3.37
1/4 in.	0.409	2.26	2.84	4.06
Hex Washer Head Screws				
Screw Designation	Washer Head Diameter in.	Thickness of member in contact with the washer, in.		
		0.0566	0.0713	0.1017
#6	0.315	1.74	2.19	3.12
#8	0.335	1.85	2.33	3.32
#10	0.399	2.20	2.77	3.96
#12	0.415	2.29	2.88	4.12
1/4 in.	0.500	2.76	3.48	4.96

Notes: Available Strengths are:
ASD: P_{nov} / Ω
LRFD: ϕP_{nov}

SECTION 4 - EXAMPLE PROBLEMS

4.1 Weld Examples

Example IV-1: Flat Section With Fillet Welded Lap Connection



Given:

1. Steel: F_y = 50 ksi, F_u = 65 ksi
2. Loads: P_{dead} = 1.0 kips, P_{live} = 3.0 kips
3. Detail of connection shown in sketch

Required:

Determine if longitudinal fillet welded connection is adequate to transmit the required strength P (ASD), and P_u (LRFD) using ASCE/SEI 7-05 load combinations.

Solution:

1. Required Strength

ASD

$$P = P_{dead} + P_{live}$$
$$= 1.0 + 3.0 = 4.0 \text{ kips}$$

LRFD

$$P_u = 1.2P_{dead} + 1.6P_{live}$$
$$= (1.2)(1.0) + (1.6)(3.0) = 6.0 \text{ kips}$$

2. Strength at Weld (Section E2.4)

L/t = 2.25/0.060 = 37.5 > 25

For L/t ≥ 25,

$$P_n = 0.75tLF_u \qquad (Eq.\ E2.4\text{-}2)$$
$$P_n = (0.75)(0.060)(2.25)(65) = 6.58 \text{ kips/weld}$$

Note: t = 0.060 in. < 0.10 in. , therefore, *Eq*. E2.4-4 does not apply.

ASD

$$\Omega = 3.05$$
$$\frac{P_n}{\Omega} = 6.58/3.05 = 2.16 \text{ kips/weld}$$
$$(2.16 \text{ kips/weld})(2 \text{ welds}) = 4.32 \text{ kips} > 4.0 \text{ kips OK}$$

LRFD

$\phi = 0.50$

$\phi P_n = (0.50)(6.58) = 3.29 \text{ kips/weld}$

$(3.29 \text{ kips/weld})(2 \text{ welds}) = 6.58 \text{ kips} > 6.0 \text{ kips}$ OK

3. Tensile strength of the 0.060 in. sheet (Section C2)

Yielding of the gross section

$$T_n = A_g F_y \qquad (Eq.\ C2\text{-}1)$$

$$= (2.50)(0.060)(50) = 7.50 \text{ kips}$$

ASD

$\Omega_t = 1.67$

$\frac{T_n}{\Omega_t} = 7.50/1.67 = 4.49 \text{ kips} > 4.0 \text{ kips}$ OK

LRFD

$\phi_t = 0.90$

$\phi_t T_n = (0.90)(7.50) = 6.75 \text{ kips} > 6.0 \text{ kips}$ OK

Rupture of the net section away from the connection

$$T_n = A_n F_u = (2.50)(0.060)(65) = 9.75 \text{ kips} \qquad (Eq.\ C2\text{-}2)$$

ASD

$\Omega_t = 2.00$

$\frac{T_n}{\Omega_t} = 9.75/2.00 = 4.88 \text{ kips} > 4.0 \text{ kips}$ OK

LRFD

$\phi_t = 0.75$

$\phi_t P_n = (0.75)(9.75) = 7.31 \text{ kips} > 6.0 \text{ kips}$ OK

Using Connection Tables

Using Table IV-1, the available strength based on sheet shear could have been determined as follows:

1. Sheet shear, Table IV-1 for two 2.25 in. fillet welds with sheet thickness = 0.060 in.

$L/t = 2.25/0.060 = 37.5 \geq 25$

$P'_n = 2.93$ kips/inch (from Table IV-1)

$P_n = (2)(2.93)(2.25) = 13.19 \text{ kips}$

ASD

$\Omega = 3.05$

$\frac{P_n}{\Omega} = 13.19/3.05 = 4.33 \text{ kips} > 4.0 \text{ kips}$ OK

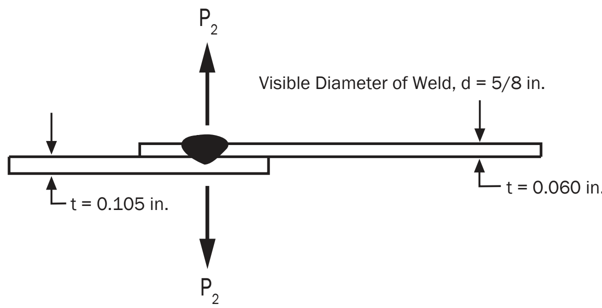
LRFD

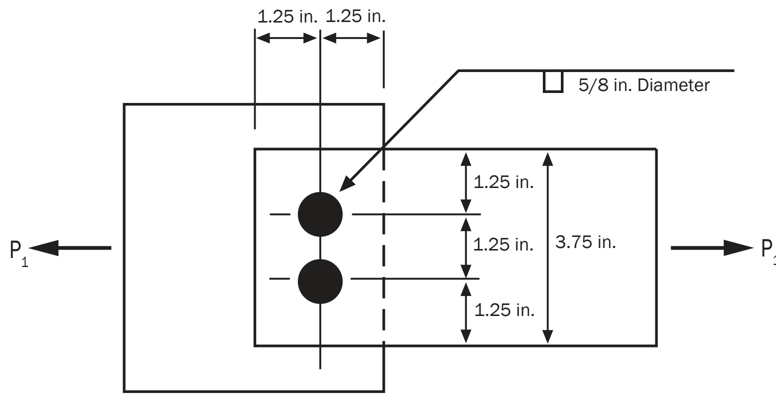
$\phi = 0.50$

$\phi P_n = (0.50)(13.19) = 6.60 \text{ kips} > 6.0 \text{ kips}$ OK

2. Check other sheet limit states as above

Example IV-2: Flat Section With Arc Spot Welded Connection





Given:

1. Steel: $F_{sy} = F_y = 33$ ksi, $F_u = 45$ ksi
2. E60 Weld Electrode, $F_{xx} = 60$ ksi
3. Detail of connection shown in sketch

Required:

Determine the available shear strength, P_1 and available tensile strength, P_2, using ASD and LRFD.

Solution:

1. Weld Dimensions

$d = 0.625$ in.

$d_a = d - t = 0.625 - 0.060 = 0.565$ in.

$d_e = 0.7d - 1.5t \leq 0.55d$ (*Eq.* E2.2.1.2-5)

$= (0.70)(0.625) - (1.5)(0.060) = 0.348$ in. < 3/8 in. N.G.

$0.55d = 0.55(0.625) = 0.344$ in. < 3/8 in. N.G.

Per Section E2.2, minimum allowable effective diameter, d_e, is 3/8 in. Weld procedures must be established and welds measured to assure that a 3/8 inch effective diameter can be consistently achieved.

2. Shear Strength, P_1

a) Strength based on weld strength (Section E2.2.1.2(a))

$$P_n = \frac{\pi d_e^{\,2}}{4} 0.75F_{xx} \quad (Eq.\ E2.2.1.2\text{-}1)$$

Using E60 electrode, F_{xx} = 60 ksi

$$P_n = \frac{\pi(0.375)^2}{4}(0.75)(60)(2) = 9.94\,\text{kips}$$

ASD

Ω = 2.55

$\frac{P_n}{\Omega}$ = 9.94/2.55 = 3.90 kips

LRFD

ϕ = 0.60

$\phi P_n = (0.60)(9.94) = 5.96$ kips

b) Strength based on sheet strength (Section E2.2.1.2(b))

d_a/t = 0.565/0.060 = 9.42

$0.815\sqrt{E/F_u} = 0.815\sqrt{29500/45} = 20.9$

Since $d_a/t < 0.815\sqrt{E/F_u}$

$P_n = 2.20 t d_a F_u$ (*Eq*. E2.2.1.2-2)

$= (2.20)(0.060)(0.565)(45)(2) = 6.71$ kips

ASD

Ω = 2.20

$\frac{P_n}{\Omega}$ = 6.71/2.20 = 3.05 kips ← CONTROLS

LRFD

ϕ = 0.70

$\phi P_n = (0.70)(6.71) = 4.70$ kips ← CONTROLS

c) Strength based on edge distance and spacing requirements (Section E2.2.1.1)

F_u/F_{sy} = 45/33 = 1.36 > 1.08

For $F_u/F_{sy} \geq 1.08$

ASD

Ω = 2.20

$e_{min} = \frac{P\Omega}{F_u t}$; therefore, (*Eq*. E2.2.1.1-1)

$$P = \frac{e_{min}F_u t}{\Omega} = \frac{(1.25)(45)(0.060)}{2.20} = 1.53\,\text{kips/weld}$$

For 2 welds; P = 3.06 kips

LRFD

ϕ = 0.70

$e_{min} = \frac{P_u}{\phi F_u t}$; therefore, (*Eq*. E2.2.1.1-2)

$P_u = e_{min}\phi F_u t = (1.25)(0.70)(45)(0.060) = 2.36$ kips/weld

For 2 welds, $P_u = 4.72$ kips

d) Edge distance shall not be less than 1.5d.

$1.5d = (1.5)(0.625) = 0.94$ in. < 1.25 in. OK

e) Clear distance between welds shall not be less than 1.0d.

$1.0d = (1.0)(0.625) = 0.625$ in.

Clear distance $= 1.250 - (2)(0.625/2) = 0.625$ in. = 0.625 in. OK

f) Clear distance between welds and end of member shall not be less than 1.0d.

$1.0d = (1.0)(0.625) = 0.625$ in.

Clear distance $= 1.250 - 0.625/2 = 0.938$ in. > 0.625 in. OK

g) Thinnest connected part, t = 0.060 in. < 0.15 in. OK

h) No weld washers required because t = 0.060 in. > 0.028 in.

i) Tensile strength of the sheet (Section C2 of Appendix A)

Yielding of the gross section

$T_n = A_g F_y$ (*Eq.* C2-1)

$= (3.75)(0.060)(33) = 7.43$ kips

ASD

$\Omega_t = 1.67$

$\frac{T_n}{\Omega_t} = 7.43/1.67 = 4.45$ kips

LRFD

$\phi_t = 0.90$

$\phi_t T_n = (0.90)(7.43) = 6.69$ kips

Rupture of the net section away from connection

$T_n = A_n F_u$ (*Eq.* C2-2)

$= (3.75)(0.060)(45) = 10.1$ kips

ASD

$\Omega_t = 2.00$

$\frac{T_n}{\Omega_t} = 10.1/2.00 = 5.05$ kips

LRFD

$\phi_t = 0.75$

$\phi_t T_n = (0.75)(10.1) = 7.58$ kips

3. Shear Strength, P_1, Using Connection Tables

Using Table IV-3 the available strength based on weld shear and sheet shear could have been determined as follows:

a) Check weld shear as in Part 2 above

$P_n = 9.94$ kips

$\frac{P_n}{\Omega}$ = 9.94/2.55 = 3.90 kips (ASD)

$\phi P_n = (0.60)(9.94) = 5.96$ kips (LRFD)

b) Shear of sheet, Table IV-3

$P_n = (3.36)(2) = 6.72$ kips

$\frac{P_n}{\Omega}$ = 6.72/2.20 = 3.06 kips (ASD) ← CONTROLS

$\phi P_n = (0.70)(6.72) = 4.70$ kips (LRFD) ← CONTROLS

c) Check edge distance and sheet as above

4. Tensile Strength, P_2

a) Check limits

1. $td_aF_u \le 3$ kips

   $(0.060)(0.565)(45) = 1.53 < 3$ kips OK

2. $e_{min} \ge d$

   1.25 in. > 0.625 in. OK

3. $F_{xx} \ge 60$ ksi OK

4. $F_u \le 82$ ksi OK

5. $F_{xx} > F_u$

   60 ksi > 45 ksi OK

b) Calculate P_n as the smaller value from *Eq*. E2.2.2-1 and *Eq*. E2.2.2-2

$$P_n = \frac{\pi d_e^2}{4} F_{xx} \qquad (Eq.\ E2.2.2\text{-}1)$$

Using E60 electrode, F_{xx} = 60 ksi

$$P_n = \frac{\pi(0.375)^2}{4}(60) = 6.63 \text{ kips per weld}$$

$$P_n = 0.8(F_u/F_y)^2\, td_aF_u \qquad (Eq.\ E2.2.2\text{-}2)$$

$$= 0.8(45/33)^2(0.060)(0.565)(45) = 2.27 \text{ kips per weld} \quad \leftarrow \text{CONTROLS}$$

ASD

Ω = 3.00

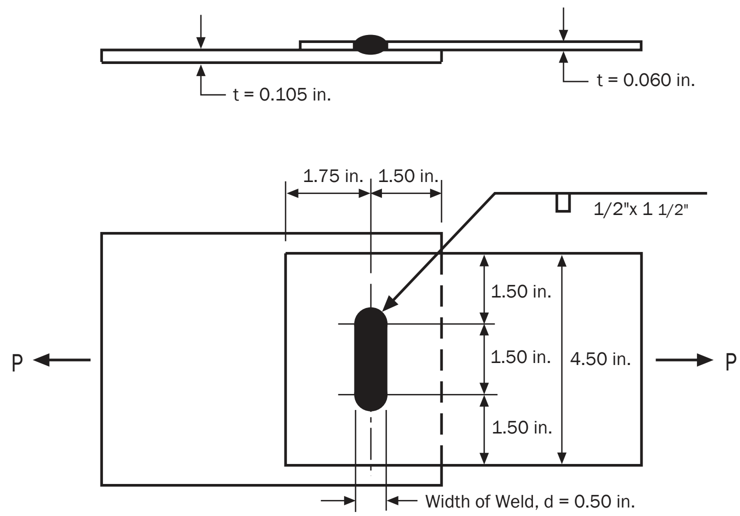
$\frac{P_n}{\Omega} = 2(2.27)/3.00 = 1.51$ kips

LRFD

ϕ = 0.50

$\phi P_n = 2(0.50)(2.27) = 2.27$ kips

Example IV-3: Flat Section With Arc Seam Welded Connection



Given:

1. Steel: F_y = 50 ksi, F_u = 65 ksi
2. E60 Weld Electrode, F_{xx} = 60 ksi
3. Total Required Strength: P = 2.90 kips (ASD), P_u = 4.10 kips (LRFD)
4. Detail of connection shown in sketch
5. Connection is in flat position as required by Section E2.3

Required:

Check the ability of the connection to transmit the required strength.

Solution:

1. Shear strength based on weld strength (Section E2.3)

Calculate P_n as the smaller value from *Eq.* E2.3-1 and *Eq.* E2.3-2.

a) Weld Metal Strength

$$P_n = \left[\frac{\pi d_e^2}{4} + Ld_e\right]0.75F_{xx} \qquad (Eq.\ E2.3\text{-}1)$$

L = 1.5 in., or maximum 3d, 3(0.5) = 1.5 in. OK

$$d_a = d - t \qquad (Eq.\ E2.3\text{-}4)$$

$$= 0.50 - 0.060 = 0.44 \text{ in.}$$

$$d_e = 0.7d - 1.5t \qquad (Eq.\ E2.3\text{-}3)$$

$$= (0.7)(0.50) - (1.5)(0.060) = 0.260 \text{ in.}$$

$$P_n = \left[\frac{\pi(0.26)^2}{4} + (1.50)(0.26)\right](0.75)(60) \qquad (Eq.\ E2.3\text{-}1)$$

= 19.9 kips

b) Base Metal Strength

$$P_n = 2.5tF_u(0.25L + 0.96d_a) \qquad (Eq.\ E2.3\text{-}2)$$

$$= (2.5)(0.060)(65)\left[(0.25)(1.5) + (0.96)(0.44)\right] = 7.77 \text{ kips} \quad \leftarrow \text{CONTROLS}$$

ASD

$\Omega \quad = 2.55$

$P_n/\Omega = 7.77/2.55 = 3.05$ kips > P = 2.90 kips OK

LRFD

$\phi \quad = 0.60$

$\phi P_n = (0.60)(7.77) = 4.66$ kips > P_u = 4.10 kips OK

2. Determine minimum edge distance in line of force (Section E2.2.1.1)

$F_u/F_y = 65/50 = 1.30 > 1.08$

For $F_u/F_y > 1.08$;

$\Omega \quad = 2.20$ (ASD)

$\phi \quad = 0.70$ (LRFD)

ASD

$$e_{min} = \frac{P\Omega}{F_u t} \qquad (Eq.\ E2.2.1.1\text{-}1)$$

for t = 0.060 in.

$$e_{min} = \frac{(2.90)(2.20)}{(65)(0.060)} = 1.64 \text{ in.} < 1.75 \text{ in. OK}$$

for t = 0.105 in.

$$e_{min} = \frac{(2.90)(2.20)}{(65)(0.105)} = 0.935 \text{ in.} < 1.5 \text{ in. OK}$$

LRFD

$$e_{min} = \frac{\overline{P}}{\phi F_u t} = \frac{P_u}{\phi F_u t} \qquad (Eq.\ E2.2.1.1\text{-}2)$$

for t = 0.060 in.

$$e_{min} = \frac{4.10}{(0.70)(65)(0.060)} = 1.50 \text{ in.} < 1.75 \text{ in. OK}$$

for t = 0.105 in.

$$e_{min} = \frac{4.10}{(0.70)(65)(0.105)} = 0.858 \text{ in.} < 1.50 \text{ in. OK}$$

Edge distance shall not be less than 1.5 d.

$1.5d = (1.5)(0.50) = 0.75$ in. < 1.50 in. OK

Clear distance between weld and end of member shall not be less than 1.0d.

$1.0d = (1.0)(0.50) = 0.50$ in.

Clear distance = 1.50 - 0.25 = 1.25 in. > 0.50 in. OK

3. Tensile Strength of the plate (Section C2 of Appendix A)

Yielding of the gross section

$$T_n = A_g F_y \qquad (Eq.\ C2\text{-}1)$$

$= (4.5)(0.060)(50) = 13.5$ kips

Ω_t = 1.67

$\frac{T_n}{\Omega_t}$ = 13.5/1.67 = 8.08 kips > 2.90 kips OK

LRFD

ϕ_t = 0.90

$\phi_t T_n = (0.90)(13.5) = 12.2$ kips > 4.10 kips OK

Rupture of the net section away from the connection

$T_n = A_n F_u$ (*Eq*. C2-2)

$= (4.5)(0.060)(65) = 17.6$ kips

ASD

Ω_t = 2.00

$\frac{T_n}{\Omega_t}$ = 17.6/2.00 = 8.80 kips > 2.90 kips OK

LRFD

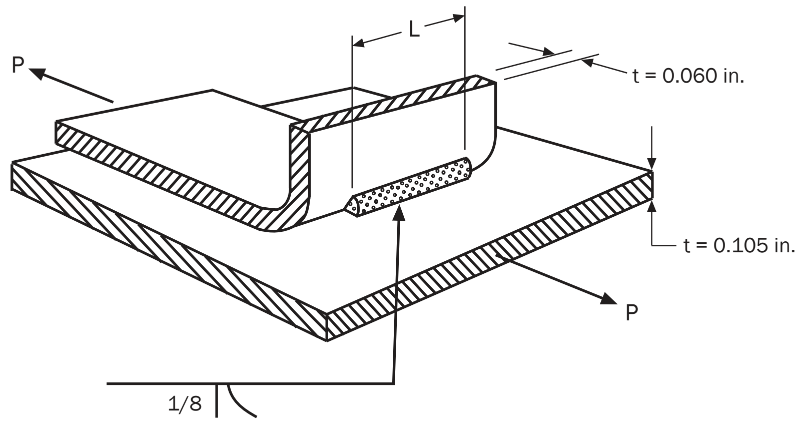
ϕ_t = 0.75

$\phi_t P_n = (0.75)(17.6) = 13.2$ kips > 4.10 kips OK

4. Final Design

Use arc seam welded connection per sketch with E60 minimum electrode.

Example IV-4: Flat Section With Flare Bevel Groove Weld



Given:

1. Steel: F_y = 50 ksi, F_u = 65 ksi
2. Nominal Loads, P_{dead} = 1.00 kips P_{live} = 3.00 kips
3. Detail of connection shown in sketch
4. Transverse loading

Required:

Determine the required weld length, L, using ASD and LRFD.

Solution:

1. Required Strength

ASD

$P = P_{dead} + P_{live}$

$= 1.00 + 3.00 = 4.00$ kips

LRFD

$P_u = 1.2\,P_{dead} + 1.6\,P_{live}$

$= (1.2)(1.00) + (1.6)(3.00) = 6.00\,\text{kips}$

2. Nominal shear strength of flare-bevel groove welds, transverse loading

$P_n = 0.833tLF_u$ (*Eq*. E2.5-1)

3. Solve for L

ASD

$\Omega = 2.55$

$P \le P_n/\Omega$ (*Eq*. A4.1.1-1)

$P \le 0.833tLF_u/\Omega$

$$\therefore L \ge \frac{\Omega P}{0.833tF_u}$$

$$L \ge \frac{(2.55)(4.00)}{(0.833)(0.060)(65)} = 3.14 \text{ in.}$$

LRFD

$\phi = 0.60$

$P_u \le \phi P_n$ (*Eq*. A5.1.1-1)

$\le \phi 0.833tLF_u$

$$\therefore L \ge \frac{P_u}{\phi 0.833tF_u}$$

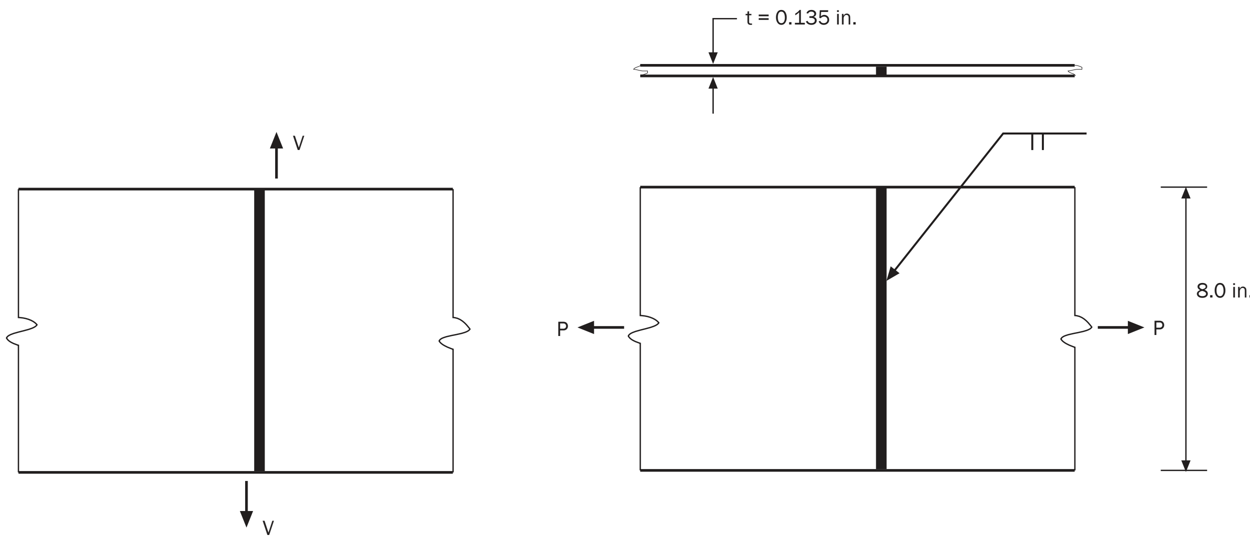
$$L \ge \frac{6.00}{(6.0)(0.833)(0.060)(65)} = 3.08 \text{ in.}$$

Eq. E2.5-4 need not be checked since t < 0.10 in.

4. Final Design

Use 1/8 inch bevel groove weld 31/4 inch long.

Example IV-5: Flat Section With Groove Welded Butt Joint



Given:

1. Steel: F_y = 50 ksi
2. E60 Weld Electrode, F_{xx} = 60 ksi
3. Detail of connection shown in sketch

Required:

1. Determine the available tensile strength normal to the effective area using ASD and LRFD.
2. Determine the available shear strength on the effective area using ASD and LRFD.

Solution:

1. Available tensile strength normal to the effective area (Section E2.1(a))

$$P_n = Lt_eF_y \qquad (Eq.\ E2.1\text{-}1)$$

$$= (8.0)(0.135)(50) = 54.0 \text{ kips}$$

ASD

$$\Omega = 1.70$$

$$\frac{P_n}{\Omega} = 54.0/1.70 = 31.8 \text{ kips}$$

LRFD

$$\phi = 0.90$$

$$\phi P_n = (0.90)(54.0) = 48.6 \text{ kips}$$

2. Available shear strength on the effective area (Section E2.1(b))

Weld Strength

$$P_n = Lt_e 0.6F_{xx} \qquad (Eq.\ E2.1\text{-}2)$$

$$= (8.0)(0.135)(0.6)(60) = 38.9 \text{ kips}$$

ASD

$$\Omega = 1.90$$

$$\frac{P_n}{\Omega} = 38.9/1.90 = 20.5 \text{ kips}$$

LRFD

$\phi = 0.80$

$\phi P_n = (0.80)(38.9) = 31.1 \text{ kips}$

Base Metal Strength

$$P_n = Lt_eF_y/\sqrt{3} \quad (Eq.\ E2.1\text{-}3)$$

$$= (8.0)(0.135)(50)/\sqrt{3} = 31.2 \text{ kips}$$

ASD

$\Omega = 1.70$

$\frac{P_n}{\Omega} = 31.2/1.70 = 18.4 \text{ kips}$

Since base metal strength governs,

$\frac{P_n}{\Omega} = 18.4 \text{ kips}$

LRFD

$\phi = 0.90$

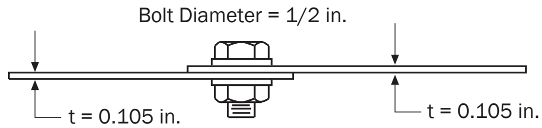
$\phi P_n = (0.90)(31.2) = 28.1 \text{ kips}$

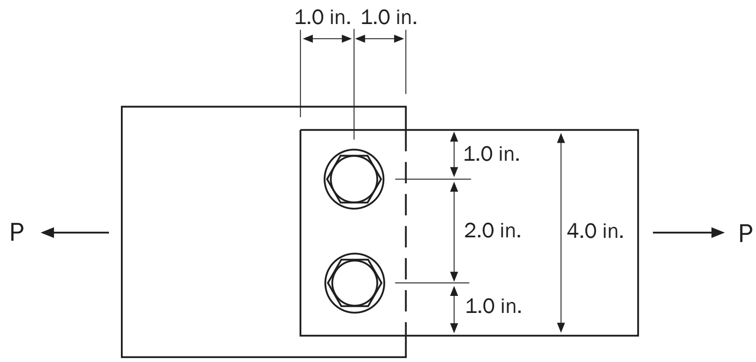
Since base metal strength governs,

$\phi P_n = 28.1 \text{ kips}$

4.2 Bolt Examples

Example IV-6: Flat Section With Bolted Connection





Given:

1. Steel: F_y = 33 ksi, F_u = 45 ksi
2. Bolts conforming to ASTM A307 with washers under bolt head and nut
3. Detail of connection shown in sketch
4. Evaluate bearing without consideration of bolt hole deformation.

Required:

Determine the ASD allowable strength, P_n/Ω, and the LRFD design strength, ϕP_n

Solution:

Thickness of thinnest part connected, t

t = 0.105 in. < 3/16 in., therefore, Section E3 applies.

1. Strength based on spacing and edge distance (Section E3.1)

$P_n = teF_u$ (*Eq*. E3.1-1)

$= (0.105)(1.0)(45) = 4.73$ kips/bolt

$F_u/F_y = 45/33 = 1.36 > 1.08$

For $F_u/F_y > 1.08$

$\Omega = 2.00$ (ASD)

$\phi = 0.70$ (LRFD)

ASD

$$\frac{P_n}{\Omega} = (2)(4.73)/2.00 = 4.73 \text{ kips}$$

LRFD

$$\phi P_n = (0.70)(2)(4.73) = 6.62 \text{ kips}$$

Distance between bolt hole centers must be $\geq 3d$.

$3d = (3)(0.50) = 1.5 \text{ in.} < 2.0 \text{ in. OK}$

Distance between bolt hole center and edge of connecting member must be $\geq 1.5d$.

$1.5d = (1.5)(0.50) = 0.75\text{ in.} < 1.0$ in. OK

Clear distance between bolt holes must be $\geq 2d$.

$2d = (2)(0.50) = 1.0\text{ in.} < (2.0 - 0.5) = 1.5$ in. OK

Clear distance between edge of bolt hole and end of member must be $\geq d$.

$d = 0.5\text{ in.} < (1.0 - 0.25) = 0.75$ in. OK

2. Strength based on tension in the sheet at or away from connection (Section C2)

Available tension strength shall not exceed the smaller value of T_n from *Eq.* C2-1 and *Eq.* C2-2.

a) Yielding of the gross cross section - Section C2(a)

$A_g = (0.105)(4.0) = 0.420\text{ in.}^2$

$T_n = A_g F_y = (0.420)(33) = 13.86\text{ kips}$ (*Eq.* C2-1)

ASD

$\Omega_t = 1.67$

$\frac{T_n}{\Omega_t} = 13.86/1.67 = 8.30$ kips

LRFD

$\phi_t = 0.90$

$\phi_t T_n = (0.90)(13.86) = 12.5\text{ kips}$

b) Rupture of the net section away from connection - Section C2(b)

Since there are no holes or other reductions in area away from the connection:

$A_n = A_g = 0.420\text{ in.}^2$

$T_n = A_n F_u = (0.420)(45) = 18.9\text{ kips}$ (*Eq.* C2-2)

ASD

$\Omega_t = 2.00$

$\frac{T_n}{\Omega_t} = 18.9/2.00 = 9.45$ kips

LRFD

$\phi_t = 0.75$

$\phi_t T_n = (0.75)(18.9) = 14.2\text{ kips}$

3. Strength based on rupture at the connection

a) Rupture of net section (Section E3.2)

A_n - based on Table E3a in Appendix A

$A_n = 0.105\left[4.0 - 2(1/2 + 1/16)\right] = 0.302\text{ in.}^2$

Since washers are provided under both bolt head and nut and there is a single line of bolts perpendicular to the direction of force:

$F_t = (0.1 + 3d/s)F_u \leq F_u$ (*Eq.* E3.2-2) where:

$d = 0.50$ in.

$s = 4.0/2 = 2.0$ in.

$F_t = \left[0.1 + (3)(0.50)/2.0\right](45)$

$= 38.25$ ksi < 45 ksi OK

$P_n = A_n F_t$ (*Eq.* E3.2-1)

$= (0.302)(38.25) = 11.55$ kips

ASD

$\Omega = 2.22$ for single shear connection

$\dfrac{P_n}{\Omega} = 11.55/2.22 = 5.20$ kips

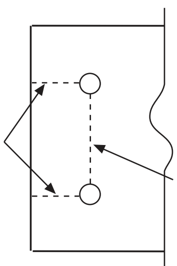
LRFD

$\phi = 0.55$ for single shear connection

$\phi P_n = (0.55)(11.55) = 6.35$ kips

b) Block shear rupture (Section E5.3)

The block shear path shown controls.



Gross area subject to shear

$A_{gv} = (2)(1.0)(0.105) = 0.210 \text{ in.}^2$

Gross area subject to tension

$A_{gt} = (2.0)(0.105) = 0.210 \text{ in.}^2$

Net area subject to shear

$A_{nv} = 0.210 - (0.105)(2)(0.5)(0.50 + 1/16) = 0.151 \text{ in.}^2$

Net area subject to tension

$A_{nt} = 0.210 - (0.105)(2)(0.5)(0.50 + 1/16) = 0.151 \text{ in.}^2$

Take R_n as the smaller value from *Eq.* E5.3-1 and *Eq.* E5.3-2.

$R_n = 0.6 F_y A_{gv} + F_u A_{nt}$ (*Eq.* E5.3-1)

$= (0.6)(33.0)(0.210) + (45.0)(0.151) = 10.95$ kips

$R_n = 0.6 F_u A_{nv} + F_u A_{nt}$ (*Eq.* E5.3-2)

$= (0.6)(45.0)(0.151) + (45.0)(0.151) = 10.87$ kips $\leftarrow$ CONTROLS

ASD

$\Omega = 2.22$ for bolted connections

$\dfrac{R_n}{\Omega} = 10.87/2.22 = 4.90$ kips

LRFD

$\phi = 0.65$

$\phi P_n = (0.65)(10.87) = 7.07$ kips

4. Strength based on bearing (Section E3.3)

Since bolt hole deformation is not a consideration, Section E3.3.1 applies.

m_f = 1.0 for single shear with washers (from Table E3.3.1-2)

d/t = 0.50/0.105 = 4.76

Since $d/t < 10$

C = 3.0 (from Table E3.3.1-1)

$P_n = m_f C d t F_u$ (*Eq*. E3.3.1-1)

$= (1.0)(3.00)(0.50)(0.105)(45) = 7.09$ kips/bolt

ASD

$\Omega = 2.50$

$\dfrac{P_n}{\Omega} = (2)(7.09)/2.50 = 5.67$ kips

LRFD

$\phi = 0.60$

$\phi P_n = (0.60)(2)(7.09) = 8.51$ kips

5. Strength based on bolt shear (Section E3.4)

$P_n = A_b F_n$ (*Eq*. E3.4-1)

$A_b = (\pi/4)(0.50)^2 = 0.196$ in.^{2}

$F_n = F_{nv} = 27.0$ ksi (from Table E3.4-1, $d \geq 1/2$ in.)

$P_n = (27)(0.196) = 5.29$ kips/bolt

ASD

$\Omega = 2.4$ (from Table E3.4-1)

$\dfrac{P_n}{\Omega} = (2)(5.29)/2.4 = 4.41$ kips

LRFD

$\phi = 0.65$ (from Table E3.4-1)

$\phi P_n = (0.65)(2)(5.29) = 6.88$ kips

6. Determine governing limit state

Comparing the values from 1, 2, 3, 4 and 5 above for ASD, the allowable design strength of the bolt shear controls:

$\dfrac{P_n}{\Omega} = 4.41$ kips

Comparing the values from 1, 2, 3, 4 and 5 above for LRFD, the design tensile strength on the net section of the connected part controls:

$\phi P_n = 6.35$ kips

Using Connection Tables

1. Using Table IV-8b the available strength based on bearing could have been determined as follows

$P_n = (7.09)(2) = 14.18$ kips

$\dfrac{P_n}{\Omega} = 14.18/2.50 = 5.67$ kips (ASD)

$\phi P_n = (0.6)(14.18) = 8.51$ kips (LRFD)

2. Using Table IV-7 the available strength based on bolt shear could have been determined as follows

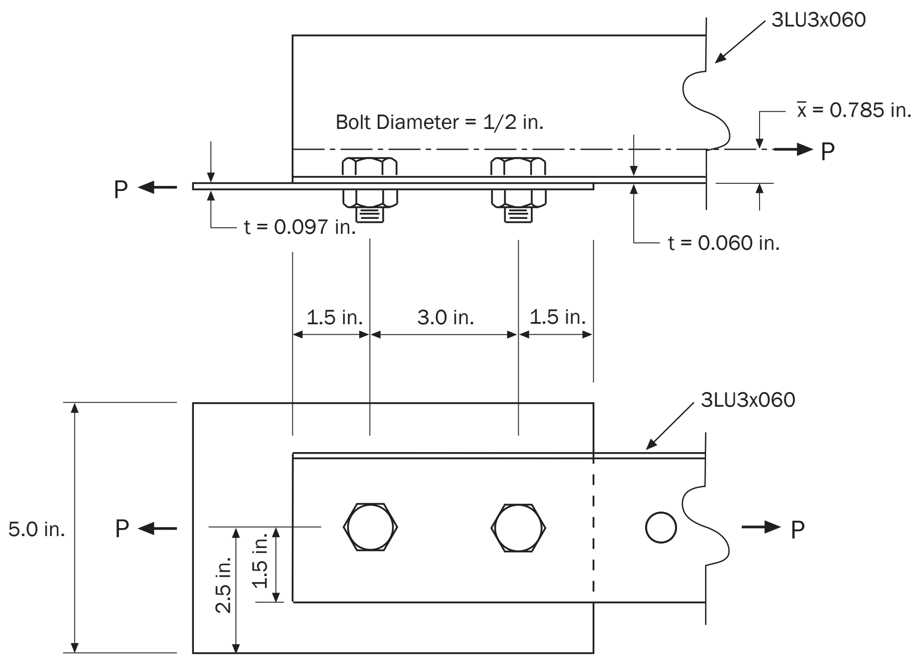
$$P_n = (5.30)(2\text{ bolts}) = 10.6\text{ kips}$$

$$\frac{P_n}{\Omega} = 10.6/2.4 = 4.42\text{ kips (ASD)}$$

$$\phi P_n = (0.65)(10.6) = 6.89\text{ kips (LRFD)}$$

3. Check net section and edge distance as above. Bolt shear controls for ASD. Tension on net section controls for LRFD.

Example IV-7: Bolted Connection with Consideration of Shear Lag



Given:

1. Steel: ASTM A653 Grade 33: F_y = 33 ksi, F_u = 45 ksi
2. 1/2 in. diameter bolts conforming to ASTM A307 in standard holes without washers under bolt head or nut
3. Section: 3LU3x060 with gross area of 0.351 in.^{2}
4. Detail of connection shown in sketch. Note unfilled hole in angle.
5. Evaluate bearing without consideration of bolt hole deformation.

Required:

Determine the ASD allowable strength.

Solution:

Calculate strength considering:

1. Shear, spacing and edge distance in the direction of applied force (Section E3.1)
2. Tensile strength of connected parts away from the connection (Section C2)
3. Rupture strength of connected parts at the connection (Sections E3.2 and E5.3)
4. Bearing strength of connected parts (Section E3.3)
5. Shear strength of bolts (Section E3.4)

1. Shear, spacing and edge distance (Section E3.1)

The thinner angle material controls by inspection.

P_n = teF_u (*Eq*. E3.1-1)

$= (0.060)(1.50)(45) = 4.05$ kips/bolt

F_u/F_y = 45/33 = 1.36

Since F_u/F_y > 1.08

Ω = 2.00 (ASD)

$\frac{P_n}{\Omega} = (2)(4.05)/2.00 = 4.05 \text{ kips}$

Distance between bolt hole centers must be $\geq 3d$.

$3d \quad = (3)(0.50) = 1.5 \text{ in.} < 3.0 \text{ in. OK}$

Distance between bolt hole center and edge of connecting member must be $\geq 1.5d$.

$1.5d = (1.5)(0.50) = 0.75 \text{ in.} < 1.5 \text{ in. OK}$

Clear distance between bolt holes must be $\geq 2d$.

$2d \quad = (2)(0.50) = 1.0 \text{ in.} < (3.0 - 0.5) = 2.5 \text{ in. OK}$

Clear distance between edge of bolt hole and end of member must be $\geq d$.

$d \quad = 0.5 \text{ in.} < (1.5 - 0.25) = 1.25 \text{ in. OK}$

2. Tensile strength of connected parts

Angle Section

a) Tension on member away from connection (Section C2)

Nominal tensile strength shall not exceed the smallest value of T_n from Section C2:

Yielding of the gross section (Section C2(a))

$T_n \quad = A_g F_y = (0.351)(33.0) = 11.58 \text{ kips}$ (*Eq*. C2-1)

$\Omega_t \quad = 1.67$

$\frac{T_n}{\Omega_t} = 11.58/1.67 = 6.93 \text{ kips}$

Rupture of the net section away from the connection (Section C2(b))

Location of unfilled bolt hole will control.

A_n - based on Table E3a in Appendix A using 1/2 inch diameter standard holes

$A_n \quad = 0.351 - (0.060)(0.500 + 1/16) = 0.317 \text{ in.}^2$

$T_n \quad = A_n F_u = (0.317)(45.0) = 14.27 \text{ kips}$ (*Eq*. C2-2)

$\Omega_t \quad = 2.00$

$\frac{T_n}{\Omega_t} = 14.27/2.00 = 7.14 \text{ kips}$

b) Rupture at the connection

Rupture of the net section (Section E3.2(c))

$P_n \quad = A_e F_u$ (*Eq*. E3.2-8)

For an angle with two or more bolts in the line of force:

$U \quad = 1.0 - 1.20\,\bar{x}/L < 0.9$ (*Eq*. E3.2-9)

$\quad = 1.0 - (1.20)(0.785/3.0) = 0.686 < 0.9 \quad \text{OK}$

$A_e \quad = UA_n$

$\quad = (0.686)(0.317) = 0.217 \text{ in.}^2$

$P_n \quad = A_e F_u$ (*Eq*. E3.2-1)

$\quad = (0.217)(45.0) = 9.77 \text{ kips}$

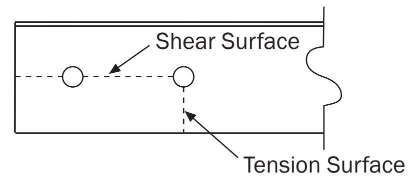
Ω = 2.22 for other than flat sheet connection

$\frac{P_n}{\Omega}$ = 9.77/2.22 = 4.40 kips

Block Shear Rupture (Section E5.3)

Gross area subject to shear

$A_{gv} = (1.5+3.0)(0.060) = 0.270 \text{ in.}^2$



Gross area subject to tension

$A_{gt} = (1.5)(0.060) = 0.0900 \text{ in.}^2$

Net area subject to shear

$A_{nv} = 0.270-(0.060)(1.5)(0.50+1/16) = 0.219 \text{ in.}^2$

Net area subject to tension

$A_{nt} = 0.0900-(0.060)(0.5)(0.50+1/16) = 0.0731 \text{ in.}^2$

Take R_n as the smaller of the values from *Eq.* E5.3-1 and *Eq.* E5.3-2.

$R_n = 0.6F_yA_{gv} + F_uA_{nt}$ (*Eq.* E5.3-1)

$= (0.6)(33.0)(0.270)+(45.0)(0.0731) = 8.64 \text{ kips} \leftarrow$ CONTROLS

$R_n = 0.6F_uA_{nv} + F_uA_{nt}$ (*Eq.* E5.3-2)

$= (0.6)(45.0)(0.219)+(45.0)(0.0731) = 9.20 \text{ kips}$

Ω = 2.22 for bolted connections

$\frac{R_n}{\Omega} = 8.64/2.22 = 3.89$ kips

Block shear controls angle tensile strength

$\frac{P_n}{\Omega}$ = 3.89 kips

Flat Sheet

a) Tension on member away from connection (Section C2)

Nominal tension strength shall not exceed the smallest value of T_n from Section C2:

Yielding on the gross section (Section C2(a))

$T_n = A_gF_y = (5.0)(0.097)(33) = 16.01$ kips (*Eq.* C2-1)

Ω_t = 1.67

$\frac{T_n}{\Omega_t}$ = 16.01/1.67 = 9.59 kips

Rupture of the net section away from the connection (Section C2(b))

Since there are no holes or other reductions in the plate away from the connection:

$A_n = A_g = (5.0)(0.097) = 0.485 \text{ in.}^2$

$T_n = A_nF_u = (0.485)(45.0) = 21.8$ kips (*Eq.* C2-2)

Ω_t = 2.00

$\frac{T_n}{\Omega_t}$ = 21.8/2.00 = 10.9 kips

b) Rupture at the connection

Rupture of the net the section (Section E3.2(a-2))

A_n - based on Table E3a in Appendix A using 1/2 inch diameter standard holes

$A_n \ = 0.097\left[5.0-(0.500+1/16)\right] = 0.430 \text{ in.}^2$

Since washers are not provided and there are multiple bolts in the line parallel to the force:

$F_t \ = F_u$ (*Eq*. E3.2-5)

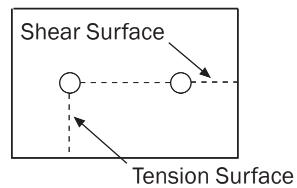
$P_n \ = A_nF_t$ (*Eq*. E3.2-1)

$= (0.430)(45.0) = 19.35 \text{ kips}$

$\Omega \ = 2.22$

$\frac{P_n}{\Omega} = 19.35/2.22 = 8.72 \text{ kips}$

Block Shear Rupture (Section E5.3)



Gross area subject to shear

$A_{gv} = (1.5+3.0)(0.097) = 0.437 \text{ in.}^2$

Gross area subject to tension

$A_{gt} = (2.5)(0.097) = 0.243 \text{ in.}^2$

Net area subject to shear

$A_{nv} = 0.437-(0.097)(1.5)(0.50+1/16) = 0.355 \text{ in.}^2$

Net area subject to tension

$A_{nt} = 0.243-(0.097)(0.5)(0.50+1/16) = 0.216 \text{ in.}^2$

Take R_n as the smaller of the values from *Eq.* E5.3-1 and *Eq.* E5.3-2.

$R_n \ = 0.6F_yA_{gv} + F_uA_{nt}$ (*Eq*. E5.3-1)

$= (0.6)(33.0)(0.437)+(45.0)(0.216) = 18.4 \text{ kips} \leftarrow$ CONTROLS

$R_n \ = 0.6F_uA_{nv} + F_uA_{nt}$ (*Eq*. E5.3-1)

$= (0.6)(45.0)(0.355)+(45.0)(0.216) = 19.3 \text{ kips}$

$\Omega \ = 2.22$ for bolted connections

$\frac{R_n}{\Omega} = 18.4/2.22 = 8.29 \text{ kips}$

Block shear controls sheet tensile strength

$\frac{P_n}{\Omega} = 8.29 \text{ kips}$

3. Bearing on connected parts (Section E3.3)

Since bolt hole deformation is not a consideration, Section E3.3.1 applies.

The thinner angle will control, by inspection

$m_f \ = 0.75$ for single shear without washers (from Table E3.3.1-2)

$d/t \ = 0.50/0.060 = 8.33$

Since $d/t < 10$

$C \ = 3.0$ (from Table E3.3.1-1)

$P_n = m_f C d t F_u$ (*Eq*. E3.3.1-1)

$= (0.75)(3.00)(0.50)(0.060)(45.0) = 3.04$ kips/bolt

$\Omega = 2.50$

$\frac{P_n}{\Omega} = (2)(3.04)/2.50 = 2.43$ kips

4. Bolt shear (Section E3.4)

$P_n = A_b F_n$ (*Eq*. E3.4-1)

$A_b = (\pi/4)(0.50)^2 = 0.196 \text{ in.}^2$

$F_n = F_{nv} = 27.0$ ksi (from Table E3.4-1, $d \geq 1/2$ in.)

$P_n = (27)(0.196) = 5.29$ kips/bolt (*Eq*. E3.4-1)

$\Omega = 2.4$

$\frac{P_n}{\Omega} = (2)(5.29)/2.4 = 4.41$ kips

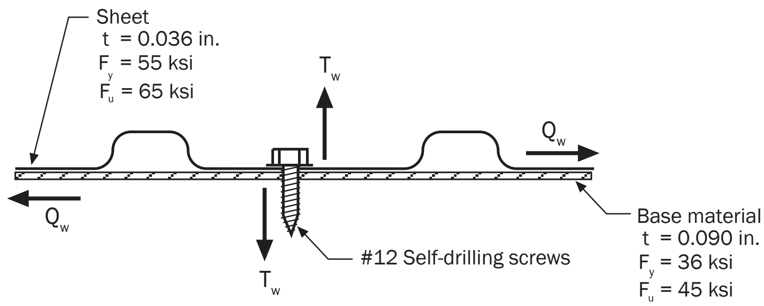
5. Determine design strength

Comparing the values from 1, 2, 3, and 4 above, the allowable strength of the bolt bearing on the angle controls:

$\frac{P_n}{\Omega} = 2.43$ kips

4.3 Screw Example

Example IV-8: Screwed Connection



Given:

1. Screws: #12 Self-drilling
   d = 0.216 in.
   d_h = 0.340 in.
   Per manufacturer's test report
   P_{ts} = 2.78 kips (screw tension strength based on tests)
   P_{ss} = 2.00 kips (screw shear strength based on tests)
2. Detail and materials as shown above
3. Wind Forces: T_w = 0.200 kips, Q_w = 0.425 kips
4. Minimum edge distance of top sheet is 0.75 inches

Required:

Verify the shear and tensile strengths using ASD and LRFD, including interaction between shear and tension.

Solution:

1. Required Strengths

ASD

Tension: $T = T_w = 0.200$ kips

Shear: $Q = Q_w = 0.425$ kips

LRFD

Tension: $\overline{T} = 1.6T_w = 1.6(0.200) = 0.320 \text{ kips}$

Shear: $\overline{Q} = 1.6Q_w = 1.6(0.425) = 0.680 \text{ kips}$

2. Shear Strength (Section E4.3)

a) Connection shear limited by tilting and bearing (Section E4.3.1)

t_1 = 0.036 in.

t_2 = 0.090 in.

t_2/t_1 = 0.090/0.036 = 2.50

For $t_2/t_1 \geq 2.5$, P_{ns} is the smaller of

$P_{ns} = 2.7t_1dF_{u1}$ (*Eq.* E4.3.1-4)

$= (2.7)(0.036)(0.216)(65) = 1.37$ kips

$P_{ns} = 2.7t_2dF_{u2}$ (*Eq.* E4.3.1-5)

$= (2.7)(0.090)(0.216)(45) = 2.36$ kips

P_{ns} = min(1.37, 2.36) = 1.37 kips

ASD

Ω = 3.00

$\frac{P_{ns}}{\Omega} = \frac{1.37}{3.0} = 0.457$ kips > 0.425 kips OK ← CONTROLS

LRFD

ϕ = 0.50

$\phi P_{ns} = (0.50)(1.37) = 0.685$ kips > 0.680 kips OK ← CONTROLS

The connection tables IV-9a and IV-9b cannot be used in this case because they assume that steels with identical F_u are used for both sheets, which is not true in this example.

b) Connection strength limited by 0.75 in. end distance (Section E4.3.2)

By inspection, the thinner sheet will govern.

$P_{ns} = teF_u$ (*Eq.* E4.3.2-1)

$= (0.036)(0.75)(65) = 1.76$ kips

ASD

Ω = 3.00

$\frac{P_{ns}}{\Omega} = \frac{1.76}{3.0} = 0.587$ kips > 0.425 kips OK

LRFD

ϕ = 0.50

$\phi P_{ns} = (0.50)(1.76) = 0.880$ kips > 0.680 kips OK

c) Shear in screw (Section E4.3.3)

$P_{ns} = P_{ss}$

= 2.00 kips

ASD

Ω = 3.00

$\frac{P_{ns}}{\Omega} = \frac{2.00}{3.0} = 0.667$ kips > 0.425 kips OK

LRFD

ϕ = 0.50

$\phi P_{ns} = (0.50)(2.00) = 1.00$ kips > 0.680 kips OK

For both ASD and LRFD, tilting and bearing govern shear strength.

3. Tensile Strength (Section E4.4)

a) Pull-out (Section E4.4.1)

$P_{not} = 0.85 t_c d F_{u2}$ (*Eq.* E4.4.1-1)

$= (0.85)(0.090)(0.216)(45)$

= 0.744 kips

Pull-over (Section E4.4.2)

$d'_w = d_h = 0.340$ in. < 0.500 in. OK

$$P_{nov} = 1.5 t_1 d'_w F_{u1} \quad (Eq.\ E4.4.2\text{-}1)$$

$$= (1.5)(0.036)(0.340)(65)$$

$$= 1.19 \text{ kips}$$

Pull-out governs

ASD

$$\Omega = 3.00$$

$$\frac{P_n}{\Omega} = \frac{0.744}{3.0} = 0.248 \text{ kips} > 0.200 \text{ kips OK} \quad \leftarrow \text{CONTROLS}$$

LRFD

$$\phi = 0.50$$

$$\phi P_n = (0.5)(0.744) = 0.372 \text{ kips} > 0.320 \text{ kips OK} \quad \leftarrow \text{CONTROLS}$$

b) Tension in screw (Section E4.4.3)

$$P_{nt} = P_{ts}$$

$$= 2.78 \text{ kips}$$

ASD

$$\Omega = 3.00$$

$$\frac{P_{nt}}{\Omega} = \frac{2.78}{3.0} = 0.927 \text{ kips} > 0.200 \text{ kips OK}$$

LRFD

$$\phi = 0.50$$

$$\phi P_{nt} = (0.50)(2.78) = 1.39 \text{ kips} > 0.320 \text{ kips OK}$$

For both ASD and LRFD, pull-out governs tensile strength.

4. Combined Strength (Section E4.5)

ASD

$$\frac{Q}{P_{ns}} + 0.71 \frac{T}{P_{nov}} \le \frac{1.10}{\Omega} \quad (Eq.\ E4.5.1\text{-}1)$$

where

$$P_{ns} = 2.7 t_1 d F_{u1} \quad (Eq.\ E4.5.1\text{-}2)$$

$$= (2.7)(0.036)(0.216)(65) = 1.36 \text{ kips}$$

$$P_{nov} = 1.5 t_1 d_w F_{u1} \quad (Eq.\ E4.5.1\text{-}3)$$

$$= (1.5)(0.036)(0.340)(65) = 1.19 \text{ kips}$$

$$\Omega = 2.35$$

$$\frac{0.425}{1.36} + 0.71 \frac{0.200}{1.19} \le \frac{1.10}{2.35} \quad (Eq.\ E4.5.1\text{-}1)$$

$$0.432 < 0.468 \text{ OK}$$

LRFD

$$\frac{\overline{Q}}{P_{ns}} + 0.71 \frac{\overline{T}}{P_{nov}} \le 1.10\phi \quad (Eq.\ E4.5.2\text{-}1)$$

where

$$P_{ns} = 2.7 t_1 d F_{u1} \qquad (\textit{Eq. } \text{E4.5.2-2})$$

$$= (2.7)(0.036)(0.216)(65) = 1.36 \text{ kips}$$

$$P_{nov} = 1.5 t_1 d_w F_{u1} \qquad (\textit{Eq. } \text{E4.5.2-3})$$

$$= (1.5)(0.036)(0.340)(65) = 1.19 \text{ kips}$$

ϕ = 0.65

$$\frac{0.680}{1.36} + 0.71 \frac{0.320}{1.19} \le (1.10)(0.65) \qquad (\textit{Eq. } \text{E4.5.2-1})$$

0.691 < 0.715 OK

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SUPPLEMENTARY INFORMATION

For Use With the
2007 Edition of the
North American Specification for the Design
of Cold-Formed Steel Structural Members

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SECTION 2 - LATERALLY UNBRACED COMPRESSION FLANGES

Previous editions of this *Manual* provided a multi-step design method for the calculation of the strength of members that will not buckle laterally while the compression flange or flanges themselves are laterally unbraced and can buckle separately by a deflection of the compression flange relative to the tension flange, accompanied by out-of-plane bending of the web and the rest of the section. An example of such a situation is the use of a hat section as a flexural member in such a manner that the "brims" are in compression. This is classified as a mode of *distortional buckling*, which is now evaluated using the provisions of the *Specification*.

The *Specification* provides both prescriptive and rational analysis methods for the consideration of distortional buckling that involves buckling of a flange and lip together. Distortional buckling modes involving out-of-plane bending of the web are not covered by the prescriptive methods; however, the Direct Strength Method provided in Appendix 1, which is permitted by *Specification* Sections C3.1.4(c) for flexural members and C4.2(c) for compression members, is generally capable of capturing this distortional mode. For this reason, the previous published design method has been removed from this edition of the *Manual*. See Example II-12 for an example of the rational analysis procedure.

SECTION 3 - TORSIONAL-FLEXURAL BUCKLING OF NON-SYMMETRICAL SHAPES

Torsional-flexural buckling of non-symmetrical sections is not covered by the *Specification*. These sections can be designed by taking F_e in Section C4 equal to σ_{TFO}.

The elastic torsional-flexural buckling stress, σ_{TFO}, is less than the smallest of the Euler buckling stresses about the x- and y- axes and the torsional buckling stress. The value of σ_{TFO} can be obtained from the following equation by trial and error:

$$\left(\frac{\sigma_{TFO}{}^3}{\sigma_{ex}\sigma_{ey}\sigma_t}\right)\alpha-\left(\frac{\sigma_{TFO}{}^2}{\sigma_{ey}\sigma_t}\right)\gamma-\left(\frac{\sigma_{TFO}{}^2}{\sigma_{ex}\sigma_t}\right)\beta-\left(\frac{\sigma_{TFO}{}^2}{\sigma_{ex}\sigma_{ey}}\right)+\frac{\sigma_{TFO}}{\sigma_{ex}}+\frac{\sigma_{TFO}}{\sigma_{ey}}+\frac{\sigma_{TFO}}{\sigma_t}=1$$

The following equation may be used for a first approximation:

$$\sigma_{TFO}=\frac{\left(\sigma_{ex}\sigma_{ey}+\sigma_{ex}\sigma_t+\sigma_{ey}\sigma_t\right)}{2\left(\sigma_{ex}\gamma+\sigma_{ey}\beta+\sigma_t\right)}-\frac{\sqrt{\left(\sigma_{ex}\sigma_{ey}+\sigma_{ex}\sigma_t+\sigma_{ey}\sigma_t\right)^2-4\left(\sigma_{ex}\sigma_{ey}\sigma_t\right)\left(\sigma_{ex}\gamma+\sigma_{ey}\beta+\sigma_t\right)}}{2\left(\sigma_{ex}\gamma+\sigma_{ey}\beta+\sigma_t\right)}$$

where

$$\sigma_{ex}=\frac{\pi^2E}{\left(KL/r_x\right)^2},\ \text{ksi}$$

$$\sigma_{ey}=\frac{\pi^2E}{\left(KL/r_y\right)^2},\ \text{ksi}$$

$$\sigma_t=\frac{1}{I_p}\left[GJ+\frac{\pi^2EC_w}{(KL)^2}\right],\ \text{ksi}$$

α = $1-\left(x_o/r_o\right)^2-\left(y_o/r_o\right)^2$

γ = $1-\left(y_o/r_o\right)^2$

β = $1-\left(x_o/r_o\right)^2$

E = modulus of elasticity = 29,500 ksi

L = unbraced length of compression member, in.

r_x = radius of gyration of cross-section about the x-axis, in.

r_y = radius of gyration of cross-section about the y-axis, in.

r_o = polar radius of gyration of cross-section about the shear center, in.

I_p = polar moment of inertia about the shear center, $\text{in.}^4 = Ar_o^2 = I_x + I_y + Ax_o^2 + Ay_o^2$

G = shear modulus = 11,300 ksi

J = St. Venant torsion constant of the cross-section, in.^4 For open sections composed of n segments of uniform thickness = $(1/3)\left(\ell_1t_1{}^3+\ell_2t_2{}^3+\ldots+\ell_nt_n{}^3\right)$

C_w= warping constant of torsion of the cross section, in.^6

ℓ_i = length of cross-section middle line of segment i, in.

t_i = wall thickness of segment i, in.

x_o = distance from shear center to centroid along the principal x-axis, in.

y_o = distance from shear center to centroid along the principal y-axis, in.

For any section, the values of x_o, y_o and C_w can be computed from the following relationships (terms are defined in Figure 3-1):

$$x_o = \frac{1}{I_x}\int_0^l w_c y t\, ds, \text{ in.}$$

$$y_o = \frac{1}{I_y}\int_0^l w_c x t\, ds, \text{ in.}$$

$$C_w = \int_0^l (w_o)^2 t\, ds - \frac{1}{A}\left[\int_0^l w_o t\, ds\right]^2, \text{ in.}^6$$

where

I_x and I_y = centroidal moments of inertia of the cross-section about the principal x- and y- axes, in.^{4}

A = total area of the cross-section, in.^{2}

t = wall thickness, in.

w_c = $\int_0^s R_c ds$, in.^{2}

w_o = $\int_0^s R_o ds$, in.^{2}

x and y = the coordinates measured from the centroid to any point P along the middle line of the cross section, in.

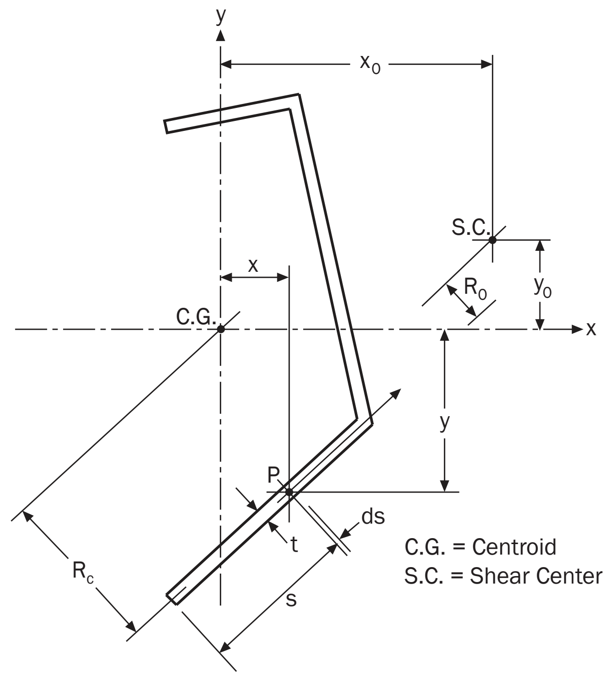

Figure 3-1
Non-Symmetric Cross-Section

s = distance measured along the middle line of the cross-section from one end to the point P, in.

l = total length of the middle line of the cross-section, in.

R_c and R_o = perpendicular distances from the centroid (C.G.) and shear center (S.C.), respectively, to the middle line at P, in. R_c or R_o is positive if a vector tangent to the middle line at P in the direction of increasing s has a counter-clockwise moment about the C.G. or S.C. as shown in Figure 3-1

SECTION 4 - SUGGESTED COLD-FORMED STEEL STRUCTURAL FRAMING, ENGINEERING, FABRICATION, AND ERECTION PROCEDURES FOR QUALITY CONSTRUCTION

The previously published "Suggested Cold-Formed Steel Structural Framing, Engineering, Fabrication, and Erection Procedures for Quality Construction" has been superseded by the AISI "Code of Standard Practice for Cold-Formed Steel Structural Framing" which provides more detailed information on many of the same topics.

The Preface to the 2006 Edition of the AISI Code of Standard Practice reads,

> *The American Iron and Steel Institute Committee on Framing Standards has developed this Code of Standard Practice for Cold-Formed Steel Structural Framing [Code of Standard Practice] to address trade practices for design, fabrication and installation of cold-formed steel structural framing products.*
>
> *This Code of Standard Practice, as revised to date, defines and sets forth accepted norms of good practice and has been developed and reviewed by a peer committee of the cold-formed steel structural framing industry. The practices defined in this Code of Standard Practice are the commonly accepted standards of custom and usage for cold-formed steel structural framing fabrication and installation. This voluntary document is intended to be used by owner's representatives, design professionals, contractors, construction managers, suppliers, manufacturers, installers and others on individual projects that utilize cold-formed steel structural framing.*
>
> *This Code of Standard Practice is not applicable to non-structural members, including but not limited to interior drywall framing, which is addressed by ASTM C645 and C754, or structural steel, structural steel joists, steel deck, metal building systems or rack structures, which are addressed by AISC, SJI, SDI, MBMA and RMI, respectively.*

The 2006 Edition of the AISI Code of Standard Practice is available from the American Iron and Steel Institute (www.steel.org) and Steel Framing Alliance (www.steelframing.org).

In 2008, the AISI Committee on Framing Standards began the process of updating the AISI Code of Standard Practice as an ANSI-approved American National Standard.

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PART VI
TEST STANDARDS

For Use With the
2007 Edition of the
North American Specification for the Design
of Cold-Formed Steel Structural Members

SECTION 1 – TEST STANDARDS

Numerical Designation of Test Standards

The numerical designation system of the test standards has changed since the previous edition of this *Manual*. The new designation has the format:

AISI S9xx-yy

where xx is an assigned integer and yy is the two digit representation of the year of acceptance or reaffirmation. Table 1 below provides a cross reference between the old designation and new designation for those test standards that were published in the previous edition of this *Manual*.

Table 1 – Test Standard Designation Cross-References

Old Designation	New Designation	Title
AISI TS-1-02	AISI S901-08	Rotational-Lateral Stiffness Test Method for Beam-to-Panel Assemblies
AISI TS-2-02	AISI S902-08	Stub-Column Test Method for Effective Area of Cold-Formed Steel Columns
AISI TS-3-02	AISI S903-08	Standard Methods for Determination of Uniform and Local Ductility
AISI TS-4-02	AISI S904-08	Standard Test Methods for Determining the Tensile and Shear Strength of Screws
AISI TS-5-02	AISI S905-08	Test Methods for Mechanically Fastened Cold-Formed Steel Connections
AISI TS-6-02	AISI S906-08	Standard Procedures for Panel and Anchor Structural Tests
AISI TS-7-02	AISI S907-08	Test Standard for Cantilever Test Method for Cold-Formed Steel Diaphragms
AISI TS-8-02	AISI S908-08	Base Test Method for Purlins Supporting a Standing Seam Roof System

User Notes and Commentary

User notes and commentary are provided in some test standards. These are included for information only, and are non-mandatory and copyrighted portions of the standards. All test standards included are American National Standards approved by the American National Standards Institute (ANSI).

Updates

In this edition, editorial changes to the test standards have been made to provide a more consistent format. Technical guidance and background information have been added as user notes and commentary.

AISI S901-08

Rotational-Lateral Stiffness
Test Method for
Beam-to-Panel Assemblies

1. Scope

1.1 The purpose of this test is to determine the rotational-lateral stiffness of beam-to-panel assemblies.

1.2 This test method applies to structural subassemblies consisting of panel, beam, and joint components, or of the joint between a wall, floor, ceiling, or roof panel and the supporting beam (purlin, girt, joist, stud, etc.).

1.3 This test method is also used to establish a limit of the displacements for avoiding joint failure.

Commentary:
This test method is used primarily in determining the strength of beams connected to panels as part of a structural assembly. The unattached "free" flange of the beam is restrained from lateral displacements and twisting by the bending stiffness of the beam elements, the connection between the "attached" flange of the beam and the panel, and the bending stiffness of the panel.

The combined stiffness, K, of the assembly determined by this test method consists of: (a) the lateral stiffness of the beam, K_a, which is a function of the geometry of the beam and geometric details of the beam-to-panel connection, (b) the local stiffness of the joint components in the immediate vicinity of the connection, K_b, which is affected by the type of fasteners, the fastener spacing used, and the geometry of the elements connected, and (c) the bending stiffness of the panel, K_c, which is a function of the moment of inertia of the panel, the beam spacing, and the beam location (edge vs. interior). The latter stiffness should be taken into account by theoretical analysis or by using the alternative test procedure described in Standard Section 13.

For specific geometric conditions, the design engineer is permitted to require duplicate testing using a new specimen with the beam orientation, or the force direction, reversed.

2. Referenced Documents

The following document or portions thereof are referenced within this Standard and shall be considered as part of the requirements of this document.

a. American Iron and Steel Institute (AISI), Washington, DC:

S100-07, *North American Specification for the Design of Cold-Formed Steel Structural Members*

b. ASTM International (ASTM), West Conshohocken, PA:

A370-07b, Standard Test Methods and Definitions for Mechanical Testing of Steel Products

E6-07b, Standard Terminology Relating to Methods of Mechanical Testing

IEEE/ASTM-SI-10-02, American National Standard for Use of the International System of Units (SI): The Modern Metric System

3. Terminology

Where the following terms appear in this Standard, they shall have the meaning as defined herein. Terms not defined in Section 3 of this Standard, AISI S100 or ASTM E6 shall have the ordinary accepted meaning for the context for which they are intended.

Subassembly. A representative portion of a larger structural assembly consisting of a wall, floor, ceiling, or roof panel with one beam connected to the panel either continuously or at regular intervals. See Figure 1.

Panel. Panel used in the subassembly, which is made of any structural material (i.e., aluminum, reinforced concrete, fiberboard, gypsum board, plastic, plywood, steel, etc.) See Figure 1.

Beam. A beam that has either an open or closed cross section. One flange of the beam is connected to the panel, and is called the "attached" flange. The other is the "unattached" flange. See Figure 1.

Joint or Connection. Local area around a mechanical fastener, weld, or adhesively bonded area that connects the beam with the panel. The local area also includes filler material such as insulation located between the panel and the beam flange.

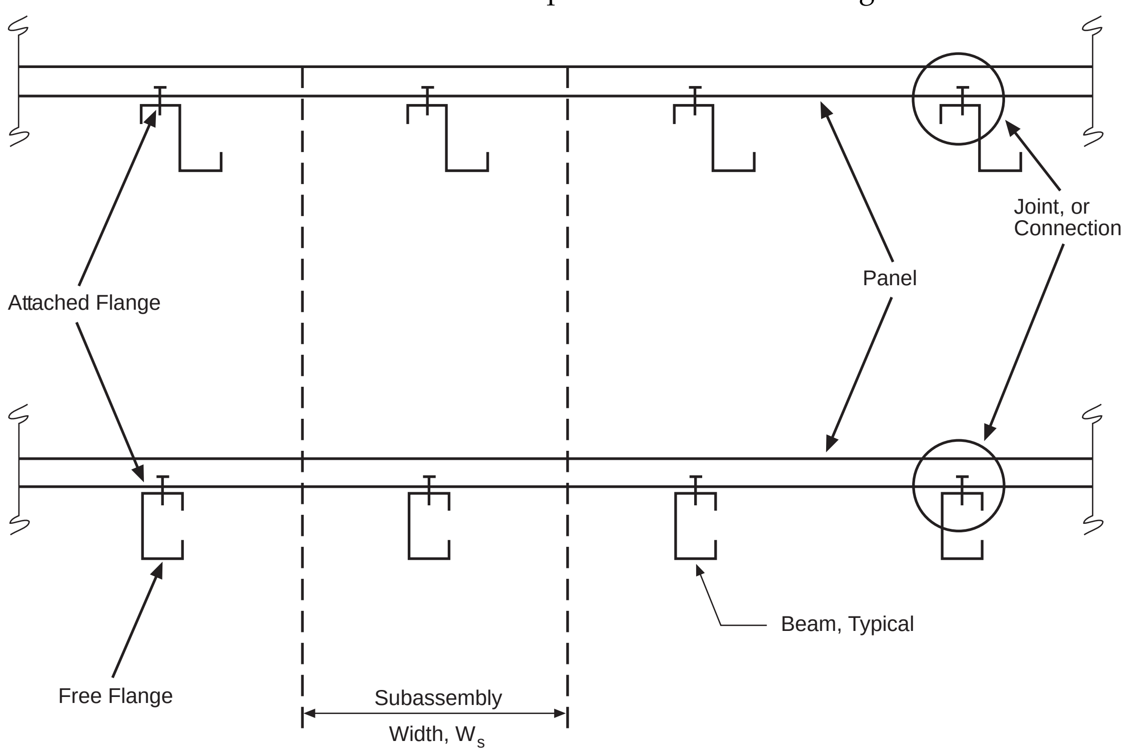
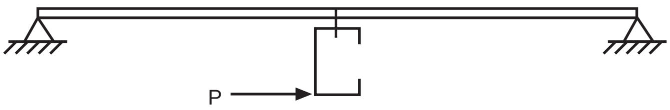


Figure 1 - Wall, Floor Ceiling or Roof Assembly

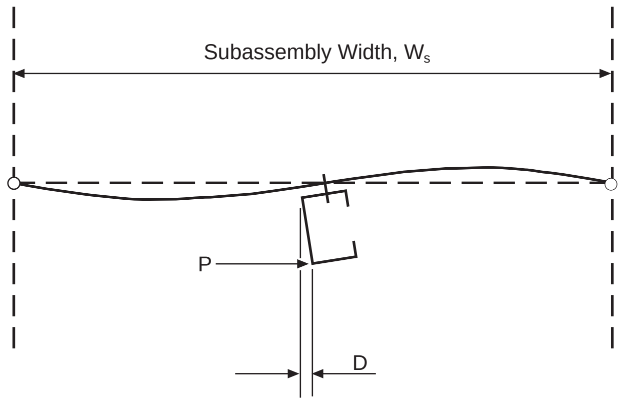
Lateral Load. Total load, P, in kips (kN), applied to the unattached flange of the beam in a plane parallel to that of the original panel position. See Figure 2.

Lateral Deflection. Lateral displacement, D, in inches (mm), of the unattached flange due to the lateral load, P. See Figure 2.

Rotational-Lateral Stiffness. Total lateral load applied on the unattached flange of the test beam, divided by the length dimension of the beam, L_B (Figure 3b), and divided by the lateral deflection of the unattached flange of the beam at that load level.



(a) Loading Diagram



(b) Deflected Subassembly

Figure 2 - Loaded and Deflected Subassembly

4. Symbols

D = Lateral displacement

D_c = Lateral displacement of unattached flange of the beam

D_N = Displacement at point N on the load-displacement curve

D_{NL} = Desired maximum lateral displacement limit

D_u = Displacement corresponding to ultimate load, P_u or ultimate displacement

E = Modulus of elasticity

F_S = Connector spacing along flange of beam. See Figure 3b.

H = Overall beam height. See Figure 3a.

H_D = Dial-gage height measured from the top of the test panel. See Figure 3a.

H_L = Height where load is applied. See Figure 3a.

I = Effective moment of inertia of panel cross section

K = Rotational-lateral stiffness

K_a = Beam stiffness

K_b = Stiffness of beam-to-panel connection

K_c = Bending stiffness of panel

K_N = Nominal test stiffness

K_t = Test stiffness

L_B = Length dimension of beam. See Figure 3b.

N = Designation of a special point on load-displacement curve which is used to determine the nominal test stiffness

P = Total lateral load applied to unattached flange
P_D = Overall panel depth. See Figure 3a.
P_N = Load at point N on the load-displacement curve
P_u = Ultimate load
W = Overall panel width. See Figure 3a.
W_C = Panel embedded distance in the support. See Figure 3a.
W_E = End dimension of test panel. See Figure 3a.
W_F = Overall width of attached beam. See Figure 3a.
W_I = Clear distance between dial-gage support and specimen support. See Figure 3a.
W_s = Width of subassembly. See Figure 1

5. Units of Symbols and Terms

Any compatible system of measurement units shall be permitted to be used in this Standard, except where explicitly stated otherwise. The unit systems considered in this Standard shall include U.S. customary units (force in kips and length in inches) and SI units (force in Newtons and length in millimeters) in accordance with IEEE/ASTM-SI-10.

6. Materials

6.1 Components of the test specimen(s) shall be measured for test analysis and records, and the component suppliers shall be identified.

6.2 Physical and material properties of the panel and beam shall be determined in accordance with AISI S100, ASTM A370, or other applicable standards.

7. Test Specimen

7.1 The overall panel width, W (Figure 3), of the specimen shall be such that the dial-gage support and the specimen support are each separated from the beam by a distance, W_I, not less than the largest of the following distances: (a) 1.5 times the overall panel depth P_D, (b) the overall width of the attached beam flange W_F, and (c) the connector spacing along the flange of the beam, F_S. For ribbed panels, W_I shall also exceed two times the width of the attached flat of the panel.

7.2 The clamped width of the specimen, W_C, shall be at least equal to two times the panel depth, but not less than 2 in. (50.8 mm).

7.3 The end dimension, W_E, shall be long enough to attach a dial gage or an extensometer to the end of the panel.

7.4 The minimum overall panel width shall be equal to:

$$W = W_E + 2W_I + W_F + W_C \tag{1}$$

7.5 The minimum beam and panel length, L_B, of the test specimen shall not be less than the largest of (a) two times the maximum connector spacing, F_S, used in field installations, or (b) the nominal coverage width of the panel. The specimen shall contain at least two fasteners in each line of connections along the beam.

7.6 Each specimen shall be assembled under the supervision of a representative of the testing laboratory, either at the manufacturer's facilities or at the testing laboratory.

7.7 Each specimen shall be assembled from new material (i.e., materials not used in previous test specimens) and in accordance with manufacturer's specifications.

7.8 The fabrication and field installation procedures specified for the overall assembly, and the tools used, shall also be used in the specimen construction.

7.9 Drilled or punched pilot holes in the panels or beams shall be the same as those used in field installations.

8. Test Setup

8.1 The test specimens shall be permitted to be tested in a horizontal or vertical position. See Figure 3 and Figure 4, respectively. The zero-load readings of the deflection-measuring device(s) shall be recorded.

8.2 The clamped end of the panel shall be the only support of the test specimen.

8.3 Where the test specimen includes a hollow-core, corrugated, or trapezoidal panel, voids of the clamped regions shall be filled with filler materials such as wood, gypsum, or similar filler materials to ensure that the clamped overall depth of the panel is maintained. For foam-filled sandwich panels, if necessary, the filler material over the distance W_C shall be permitted to be replaced with wood, gypsum, or similar filler materials.

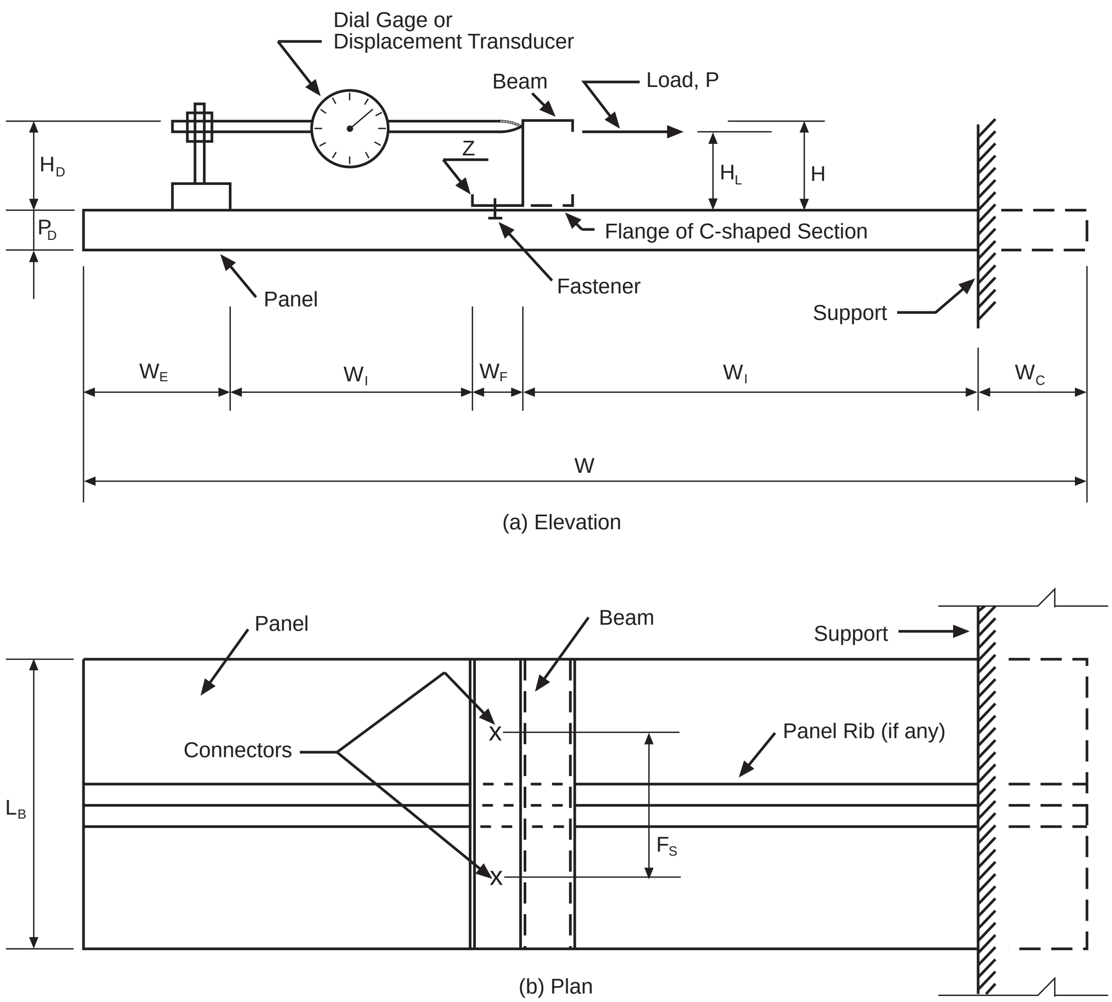


Figure 3 - Test Specimen and Horizontal Test Setup

8.4 Loads applied to the unattached flange shall be introduced at the extreme fiber of the beam, or at the intersection of the outer faces of the unattached flange and the web.

8.5 If the beam does not have a flat face perpendicular to the panel at the locations where the load is to be applied and the lateral displacement is to be measured, brackets shall be mechanically attached to the beam web to provide a flat surface. Figure 5 shows a typical application of a load bracket and/or dial gage bracket. The attachment of either bracket shall be accomplished such that the bracket does not stiffen the beam, or reduce its distortion.

8.6 The total lateral load applied, P, shall be distributed over several locations, if necessary, to reduce variations in the lateral deflection along the length of the unattached flange.

8.7 The load application shall be accomplished by chain or wire. The direction of the applied load shall remain parallel to the original plane of the panel. See Figure 5.

8.8 One or more dial gages or displacement transducers shall be used to measure the lateral displacements during loading. The gages shall be arranged symmetrically about the mid-width point, and have graduations at not greater than 0.001 in. (0.0254 mm) intervals.

9. Test Procedure

9.1 The dial-gage height, H_D, and load height, H_L, as shown in Figure 3, shall be adjusted such they are equal to or as close as possible the overall beam depth, H. Prior to loading the test specimen, the dimensions, H_D and H_L, and the dial-gage readings shall be recorded.

9.2 No preload shall be used. The load shall be applied in a direction that is for the intended use of the system.

9.3 The applied load shall be increased in five or more equal increments to the maximum expected value, in order to produce deflection increments of not more than 5 percent of the beam depth.

9.4 If the specimen includes fiberglass insulation or other non-metallic elements in the joint between panel and beam, the load shall be held at each increment for 5 minutes before reading the lateral movement.

9.5 After each load increment is added, and the deflection has stabilized, the load and lateral movement of the unattached flange shall be measured and recorded.

9.6 A test shall be terminated at failure (fastener pullout, fastener failure, panel buckling, panel failure, beam failure, etc.) and the mode of failure recorded, unless the design engineer has determined that the application of the rotational-lateral stiffness, K, occurs at lower load or displacement levels and that the test shall be permitted to be terminated earlier.

10. Number of Tests

10.1 The minimum number of tests for one set of parameters shall be three. For parametric studies using multiple values of one or more parameters, a smaller number of tests shall be permitted to be used.

10.2 If used as part of a series of at least three tests, one test shall be permitted to be sufficient for a specific condition of an all-metallic mechanically-fastened specimen using the same basic components, but using unique geometrical or physical-property differences such as fastener spacings, different beam or panel yield stresses, etc.

10.3 Three tests shall be required for any specific condition of welded or adhesively-bonded specimens, or for specimens using non-metallic materials.

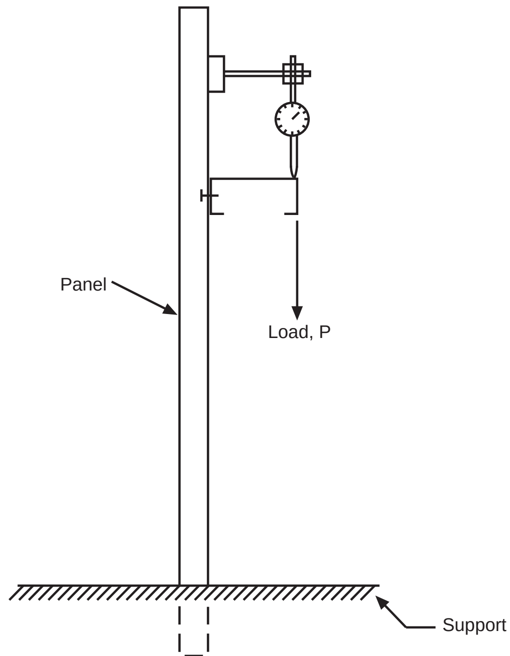


Figure 4 - Vertical Test Setup

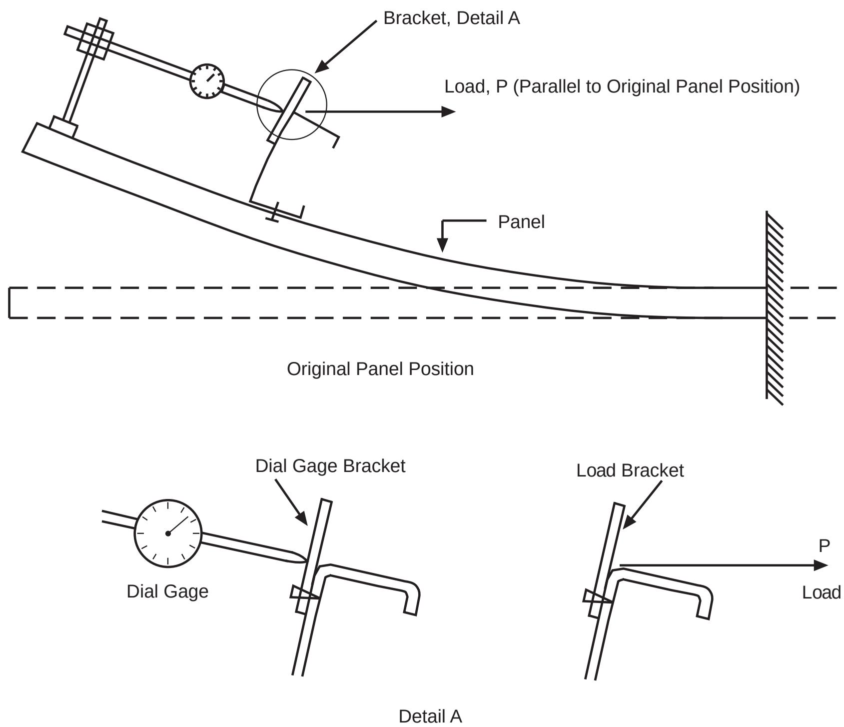


Figure 5 - Dial Gage and Load Bracket

10.4 When the rotational-lateral stiffness for three or more panel or beam thicknesses with otherwise identical parameters is to be determined, at least two specimens each with the minimum and the maximum thickness shall be tested. For a ratio of maximum-to-minimum thicknesses greater than 2.5, additional specimens with intermediate thicknesses shall be tested. One test of every thickness shall be permitted to be used in accordance with Section 10.2.

10.5 When the rotational-lateral stiffness for a range of screw spacings is to be determined, the minimum number of specimens shall be in accordance with this section. One test of every screw spacing shall be permitted to be used in accordance with Section 10.2.

10.5.1 For a ratio of maximum-to-minimum screw spacings equal to or less than 2, at least two specimens each with the minimum and the maximum screw spacing shall be tested.

10.5.2 For a range of five or more different screw spacings, or for a ratio of maximum-to-minimum screw spacings greater than 2, additional specimens with intermediate spacings must be tested.

10.6 Where the rotational-lateral stiffness for a range of other panel parameters (i.e., yield stress or ultimate strength, changes in geometry, etc.) are to be determined, a number of tests similar to the requirements under Sections 10.2 through 10.5 shall be performed.

10.7 For unsymmetric or staggered fastener arrays and/or beams unsymmetric about a plane parallel to the web, duplicate tests shall be permitted to be required by the design engineer using new specimens with the beam orientation, or the force direction, reversed.

11. Test Evaluation

11.1 Typical load-displacement curves (P vs. D) obtained from the tests shall be graphed such as shown in Figure 6. For multiple tests of one set of test parameters, the curve resulting in the lowest value of K_t, as defined in Section 11.2, shall be used for the test evaluation procedure.

Commentary:
The test stiffness, K_t, includes the stiffness effects of the beam, K_a, and the beam-to-panel connection, K_b, but excludes the bending stiffness of the panel, K_c, and follows the relationship as follows:

$K_t = (1/K_a+1/K_b)^{-1}$

11.2 The test stiffness, K_t, at any load level shall be determined as follows:

$$K_t = (P/D)/L_B \quad (2)$$

11.3 The nominal test stiffness, K_N, shall be determined as follows:

$$K_N = (P_N/D_N)/L_B \quad (3)$$

where P_N and D_N are defined by (a) through (c) based on the shape of the load-displacement (P-D) curve:

(a) When the load reaches the ultimate load, Pu, prior to displacement reaching its ultimate, Du, as shown in Figure 6(a):

$$P_N = 0.8P_u \quad (4)$$

D_N =Displacement at P_N

(b) When displacement reaches the ultimate displacement, D_u, prior to the load reaching its ultimate load, P_u, as shown in Figure 6(b):

$$D_N = 0.8D_u \tag{5}$$

P_N =Load at D_N

(c) When a P-D curve changes from bending upward to bending downward, as shown in Figure 6(c), a tangent is drawn from the origin to the P-D curve at point N, such that

$$D_N \leq 0.8D_u \tag{6}$$

$$P_N \leq 0.8P_u \tag{7}$$

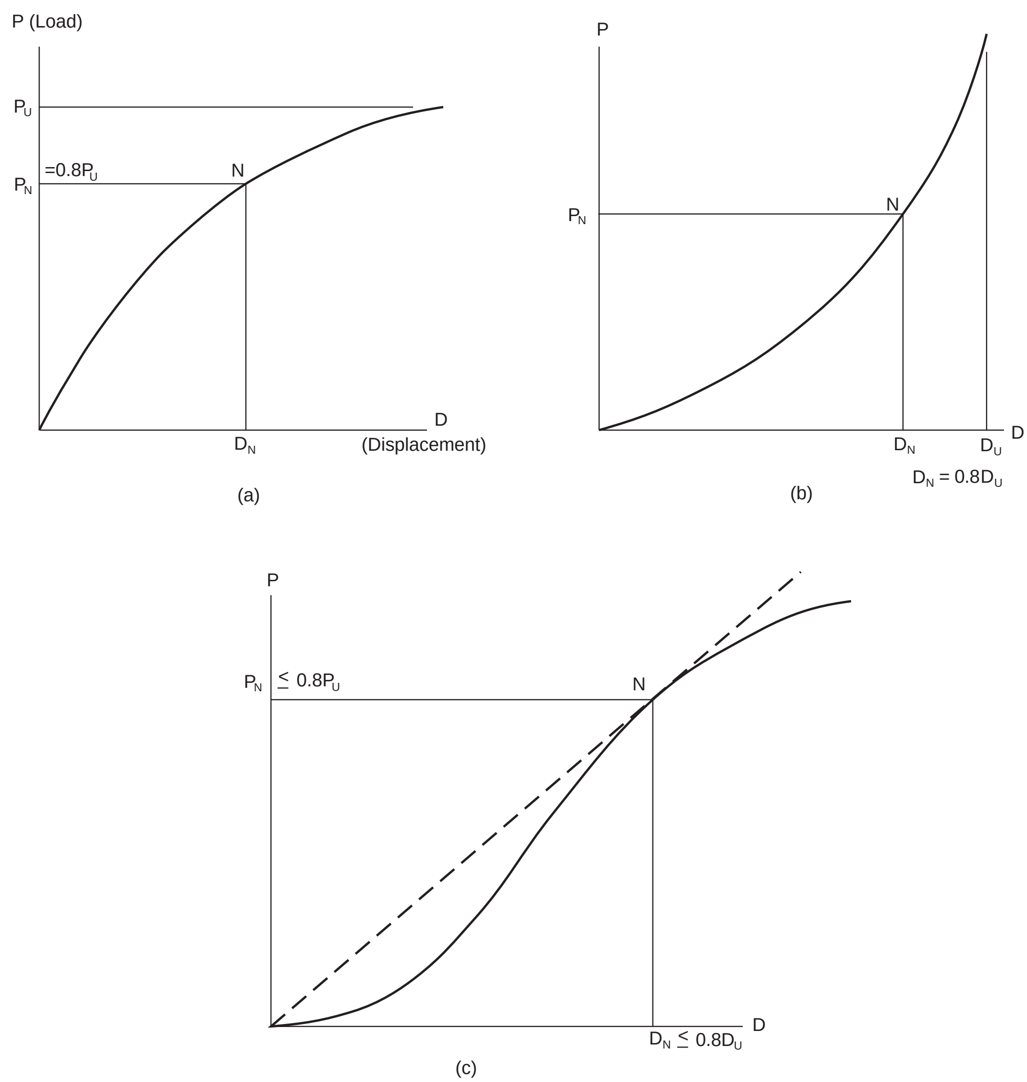


Figure 6 - Typical Load Displacement Curves

11.4 When the design engineer specifies in advance a desired maximum lateral displacement limit of D_{NL}, the test shall be permitted to be discontinued when D_{NL} is reached, and K_N shall be permitted to be determined from P_N at D_{NL}, as long as the limits under Section 11.3 are observed and D_{NL} is not exceeded in design applications.

11.5 Where either H_D or H_L is not equal to the overall beam height, H, K_t and K_N shall be corrected by the factor H_DH_L/H^2.

11.6 In addition, K_t and K_N shall be adjusted by the stiffness contributions of the panel, K_c, derived from the linear-elastic displacement analysis representing the design applications, unless such an analysis shows that these contributions are insignificant. Alternately, the panel stiffness shall be included by using the alternative test method under Section 13.

11.7 For subassemblies such as shown in Figure 2, where the applied lateral test loads cause a bending moment distribution in the panel similar to that shown in Figure 7, a lateral displacement, D_c, of the unattached flange of the beam shall be determined as follows:

$$D_c = \frac{PH_L^2 W_s}{12EI} \tag{8}$$

where W_s is the width of the subassembly as shown Figure 2 and Figure 7, E is the modulus of elasticity of the panel material, and I is the effective moment of inertia of the panel cross section (obtained from deflection determination calculations for cold-formed metal deck panels.)

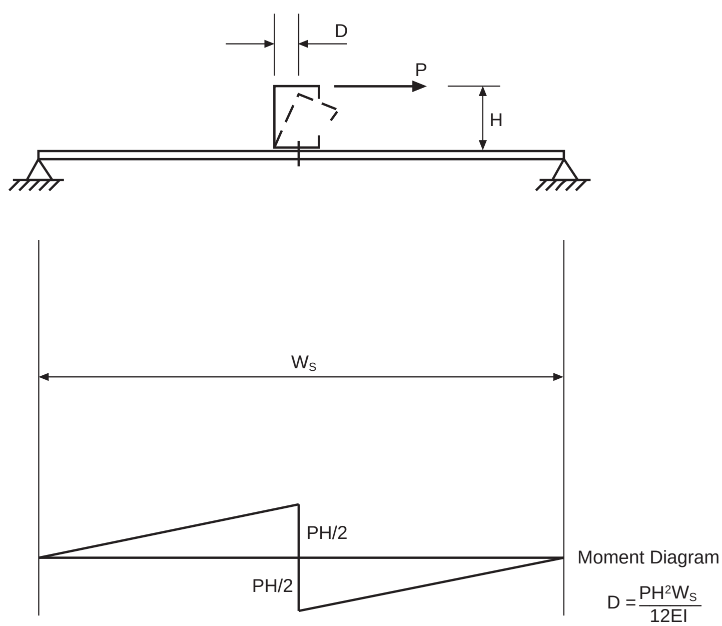


Figure 7 – Bending Moment Diagram with an Interior Beam

The panel stiffness shall be determined as follows:

$$K_c = 1/D_c \tag{9}$$

11.8 The overall rotational-lateral stiffness of the assembly shall be determined as follows:

$$K = \left(\frac{1}{K_t} + \frac{1}{K_c}\right)^{-1} \tag{10}$$

11.9 When tests covering ranges of parameters (thickness, yield stresses, screws spacings, etc.) are conducted according to Section 10, a linear interpolation shall be permitted to be used to determine intermediate K values.

12. Test Report

12.1 The test report shall consist of a description of all specimen components, including drawings defining the actual and nominal geometry, material specifications, material properties test results describing the physical properties of each component, and the supply sources. Differences between the actual and the nominal dimensions and material properties shall be noted in the report.

12.2 The test report shall contain a sketch or photograph of the test setup, the latest calibration date and accuracy of the equipment used, the signature of the person responsible for the tests, and a tabulation of all raw and evaluated test data.

12.3 All graphs resulting from the test evaluation procedure shall be included in the test report.

12.4 A summary statement, or tabulation, shall be included in the summary of the report to define the actual and nominal rotational-lateral stiffness derived from the tests conducted, including all limitations.

13. Alternative Rotational-Lateral Stiffness Test*

This alternative rotational lateral stiffness test method provided in Section 13 shall be permitted to be used in place of the basic test methods, as provided in Sections 7 through 11.

13.1 To include the panel-stiffness contribution in the test, rather than by linear-elastic analysis, the design engineer shall be permitted to request a test specimen and setup as shown in Figure 8 and Figure 9, respectively.

13.2 The test specimens shall be as described under Section 7 except as permitted in Section 13.2.1 through Section 13.2.4.

13.2.1 The minimum overall panel width of the specimen, W (Figure 8), shall be equal to

$$W = W_E + W_I + W_C \tag{11}$$

13.2.2 The minimum end dimension, W_E, shall equal the width of the attached beam flange plus 4 in. (102 mm) to allow the development of local deformation patterns around the fasteners as they would develop in a real structure.

* This method is conservative as compared to the basic methods which analytically account for the stiffness of the panel.

13.2.3 For specimens representing interior-beam subassemblies, as shown in Figures 1 and 2, the dimension W_I of the test specimen (Figure 8) shall be equal to $^1/_{12}$ of the subassembly width, W_S (Figures 1 and 2), to assure that the overall rotational-lateral stiffness contribution of the test-specimen panel is the same as that of the subassembly.

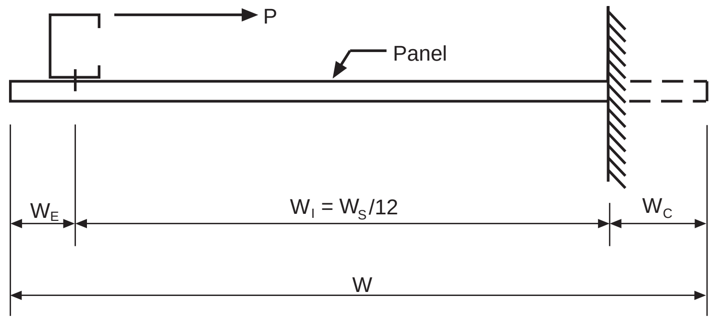


Figure 8 - Panel Width for Alternative Test

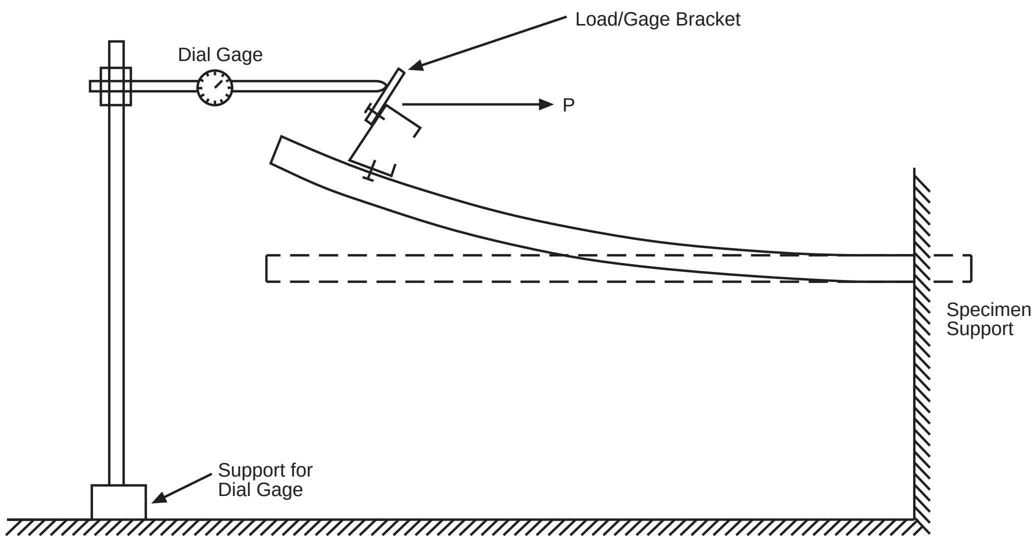


Figure 9 - Test Setup for Alternative Test

13.2.4 For other subassembly conditions, W_I shall be based upon the field conditions.

13.3 The test-setup shall be as described under Section 8 except as permitted in Section 13.3.1 and Section 13.3.2.

13.3.1 The clamped support shown in Figures 8 and 9 shall minimize the rotation and translation of the test specimen at the support.

13.3.2 The lateral-displacement measuring device shall be located on a support fixed relative to the clamped support of the test panel, as shown in Figure 9.

13.4 Test procedures shall be the same as described under Section 9.

13.5 The number of tests shall be determined as described in Section 10.

13.6 The test-evaluation procedure shall follow the underlying principles used to develop Section 11. The test stiffness at any load level shall be determined according to Equation 2 and the nominal test stiffness shall be determined according to Equation 3.

13.7 For other interior-beam spacings, for exterior-beam conditions, or for other geometrical conditions, the measured displacements shall be adjusted by a linear-elastic analysis to represent the field conditions, unless such an analysis shows that these displacements and their effect on K are insignificant.

AISI S902-08

Stub-Column Test Method for Effective Area of Cold-Formed Steel Columns

1. Scope

1.1 This test method provides methodology to determine the effective cross-sectional area of cold-formed steel columns.

1.2 This test method provides requirements for testing, and equations to determine, the effective area of a cold-formed column section at ultimate load, A_{euN}, and the load- or stress-dependent effective area, A_e.

1.3 The test method provides a means to observe, measure, and account for local buckling deformations when the appearance of a column section under stress must be determined.

1.4 The determination of A_e is conducted by either of the following methods:

(1) The basic method provided in Sections 5 through 11, or

(2) An alternative method provided in Appendix A.

2. Referenced Documents

The following documents or portions thereof are referenced within this Standard and shall be considered as part of the requirements of this document.

a. American Iron and Steel Institute (AISI), Washington, DC:

S100-07, *North American Specification for the Design of Cold-Formed Steel Structural Members*

b. ASTM International (ASTM), West Conshohocken, PA:

A370-07b, Standard Test Methods and Definitions for Mechanical Testing of Steel Products

E6-07b, Standard Terminology Relating to Methods of Mechanical Testing

IEEE/ASTM-SI-10-02, American National Standard for Use of the International System of Units (SI): The Modern Metric System

3. Terminology

Where the following terms appear in this Standard they shall have the meaning as defined herein. Terms not defined in Section 3 of this Standard, AISI S100 or ASTM E6 shall have the ordinary accepted meaning for the context for which they are intended.

Elements. Straight or curved portions of the cross section of a column or stub-column.

Local Buckling. The local buckling mode of a flat element of a column cross section, which influences the overall column-buckling behavior.

Overall Buckling. Buckling of a column as a function of its overall length.

Stub-Column. An axial compression member of the same cross section and material as the column for which the strength needs to be determined, but short enough to preclude overall column buckling, if possible.

4. Symbols

A = Gross cross-sectional area of a column without holes or perforation, or the minimum cross-sectional area of a column with holes or perforation

A_a = Average of all gross cross-sectional areas of the stub-columns without holes or perforations in a test unit, or the average minimum cross-sectional areas of the stub-columns with holes or perforations in a test unit

A_e = Effective cross-sectional area of a stub-column at a load less than the ultimate test load, or the effective area of a full-length column

A_{ei} = Effective cross-sectional area of a stub-column at load P_i

A_{eu} = Effective cross-sectional area of a stub-column at ultimate load

A_{eua} = Average effective cross-sectional area of a test unit of stub-columns at the ultimate axial load

A_{euN} = Nominal effective cross-sectional area at ultimate load adjusted to the nominal thickness and the minimum specified yield stress

A_{eu1} = Effective cross-sectional area of a stub-column with parameters of Test Unit 1 at ultimate load

A_{eu2} = Effective cross-sectional area of a stub-column with parameters of Test Unit 2 at ultimate load

A_N = Nominal gross cross-sectional area of a stub-column

A_1 = Minimum gross cross-sectional area of a stub-column with parameters of Test Unit 1 at ultimate load

A_2 = Minimum gross cross-sectional area of a stub-column with parameters of Test Unit 2 at ultimate load

D = Axial shortening of a stub-column at load P

D_i = Axial shortening of a stub-column at load P_i

D_u = Axial shortening of a stub-column at load P_u

f = Average axial stress assumed to be uniformly distributed over the effective cross-sectional area A_e

f_i = Average axial stress assumed to be uniformly distributed over the effective cross-sectional area, Aei at load P_i

f_o = Average axial stress assumed to be uniformly distributed over the effective cross-sectional area, Ae, above which the section is not fully effective

F_n = Nominal ultimate stress, assumed to be uniformly distributed over the effective cross section of a column as calculated from Section C4 of AISI S100, at which flexural, torsional, torsional-flexural, distortional, or local buckling, and/or yielding, can occur

F_u = Ultimate stress, assumed to be uniformly distributed, at which local failure occurs in a tested stub-column

F_y = Minimum specified elastic limit or yield stress of column or stub-column material

F_{ya} = Average elastic limit or yield stress of the sheet steel for a given test unit

F_{yi} = Individual elastic limit or yield stress of the sheet-steel specimens in a test unit

F_{yN} = Minimum specified elastic limit or yield stress of column or stub-column material

i = Load-displacement reading number for a particular stub-column test (load displacement D_i at load P_i)

j = Total number of load-displacement readings taken for a particular stub-column test

L = Length of the stub-column test specimen

L_p = Pitch of a repeating pattern of perforations along the longitudinal column axis
n = Ratio of the effective cross-sectional area at the ultimate load to the full cross-sectional area, A_{eu}/A
P = Applied axial compression force (column load)
P_i = Applied load at load increment i
P_N = Nominal failure load of a column
P_u = Ultimate stub-column load at which local failure occurs
P_{ua} = Average of all ultimate stub-column loads within a test unit
r = Minimum radius of gyration of the cross-sectional area, A
t = Nominal base-steel thickness exclusive of coating
t_a = Average of all base-steel thicknesses within a test unit, exclusive of coating
t_N = Nominal base-steel thickness within a test unit exclusive of coating
W = Greatest overall width of the cross section including corner(s)

5. Units of Symbols and Terms

Any compatible system of measurement units shall be permitted to be used in this Standard, except where explicitly stated otherwise. The unit systems considered in this Standard shall include U.S. customary units (force in kips and length in inches) and SI units (force in Newtons and length in millimeters) in accordance with IEEE/ASTM-SI-10.

6. Test Fixture

6.1 Linear displacement devices for measuring lateral displacements shall have a 0.001 in. (0.0254 mm) least-reading capability.

6.2 Measuring devices for determination of the actual geometry of a test specimen shall have a 0.001 in. (0.0254 mm) least-reading capability.

6.3 If axial shortening is recorded, the measuring device shall have a 0.0001 in. (0.00254 mm) least-reading capability.

User Note:
It is recommended that ASTM E4-07, Standard Practices for Force Verification of Testing Machines, be used as applicable.

7. Test Unit

7.1 A test unit shall include a minimum of three identical stub-column specimens and a minimum of two corresponding sheet-type tensile specimens.

7.2 The specimens within a unit shall represent one type of cold-formed steel section with the same specified geometrical, physical, and chemical properties. The specimens shall be permitted to be taken from the same column or from different production runs provided the source of the specimens is properly identified and recorded.

7.3 If stub-column specimens are taken from different production runs, at least two corresponding sheet-type specimens shall be taken and tested from each production run.

7.4 The stub-column test specimens shall be used to determine:

(1) The actual geometry of each specimen

(2) The ultimate stub-column test load

(3) Axial shortening at each load level if the alternative test-evaluation method described in Appendix A is used

(4) Lateral displacements of the specimen at locations of interest (if desired)

7.5 The tensile test specimens shall be used to define the yield stress of each stub column specimen in accordance with the requirements of ASTM A370.

7.6 For each test specimen and test unit, the measured geometrical and tested physical properties of the individual specimens shall meet the requirements stated by the fabricator and material producer, respectively.

7.7 If the average area, thickness, or yield stress of a test unit varies by more than 20 percent from the respective nominal or specified-minimum value, the test unit shall be considered to be non-representative of the column section, and further evaluation of the effective area shall be considered to be invalid.

8. Test Specimen

The stub-column specimens shall meet length and end-flatness requirements as follows, depending on whether unconnected or welded endplates are used.

8.1 Stub-Column Length. The length requirements of the stub-column test specimen, L, as shown in Figures 1 and 2, shall be as follows: (1) short enough to eliminate overall column buckling effects, and (2) sufficiently long to minimize the end effects during loading, which means that its center portion be representative of the repetitive hole pattern in the full column.

8.1.1 To eliminate overall column-buckling effects, the stub-column length shall not exceed twenty times the minimum radius of gyration, r, of the cross section, A, except where necessary to meet the requirements of Sections 8.1.2 through 8.1.5.

8.1.2 For unperforated columns (Figure 1a) the stub-column length shall not be less than three times the greatest overall width of the cross section, W.

8.1.3 For perforated columns in which the pitch (gage length) of the perforation pattern, L_p, for a single hole or a group of holes, is smaller than, or equal to, the greatest overall width, W, of the cross section (Figures 1b and 1g), or for a single hole pattern with a gage length larger than the greatest overall width (Figure 1c), the specimen length shall not be less than three times the greatest overall width of the cross section, W. For widely spaced hole patterns (Figure 1c), the significant hole or hole pattern shall be located at or near the mid-length of the stub column.

8.1.4 For perforated columns in which the pitch of the perforation pattern, L_p, is greater than the widest side, W, of the cross section (Figures 1d, 1e, 1f, and 1h), the specimen length shall not be less than three times the pitch of the perforation pattern.

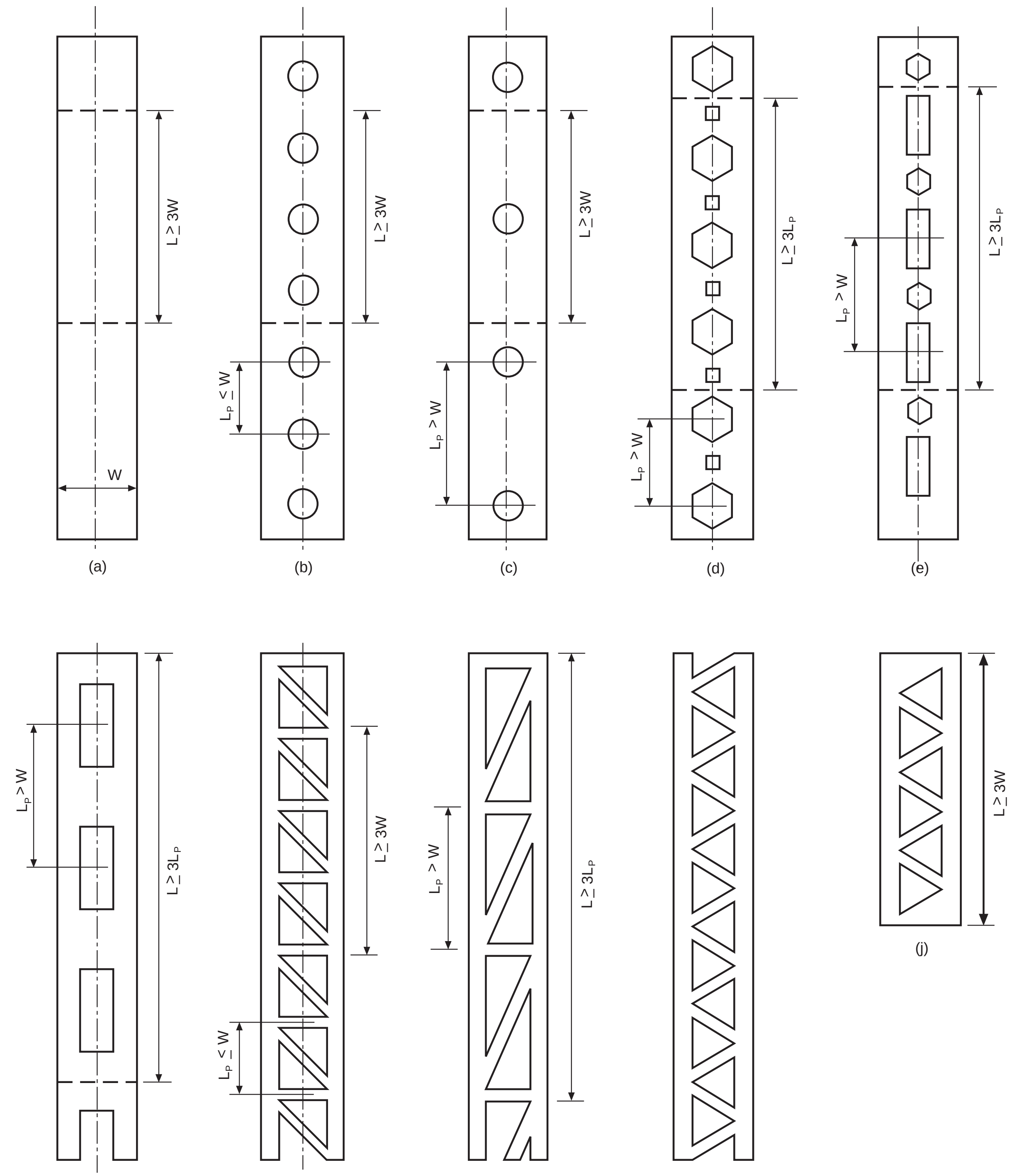


Figure 1 – Hypothetical Perforation Patterns and Suggested Stub Column Lengths

NOTES: (1) Perforations shown are in a flat portion of a member with width W

(2) L = Length of stub-column

(3) L_p = Pitch Length of Perforation Pattern

8.1.5 For perforated sections in which the specimen end planes must pass through the normal perforation pattern (Figure 1i), a special section (Figure 1j) shall be permitted to be fabricated to obtain full cross-sectional surfaces at the specimen ends.

8.2 Stub-Column End Surface Preparation. The end planes of the stub-column test specimens shall be carefully cut to a flatness tolerance of plus or minus 0.002 in. (0.0508 mm). When the required flatness can be achieved, welding of the stub-column ends to the endplates shall not be required. However, when this flatness cannot be achieved, steel endplates shall be continuously welded to both ends of the specimen so that there is no gap between the ends of the stub column and the endplates.

8.3 Stub-Column Specimen Source. Stub-column test specimens shall be permitted to be cut from the commercially fabricated column product. Alternatively, stub columns shall be permitted to be specially fabricated provided care is taken not to exceed the cold work of forming expected in the commercial product; however, subsequent proof tests using specimens from commercially produced columns shall be considered.

8.4 Tensile Specimen Source. Longitudinal tensile specimens shall be cut from the center of the widest flat of a formed section from which the stub-column specimens have been taken. If perforations are large and frequent in all flats of the formed section, the tensile specimens shall be permitted to be taken from the sheet or coil material used for the fabrication of the stub-column specimens. The tensile specimens shall not be taken from parts of a previously tested stub column.

8.5 Endplate Requirements. Steel endplates shall be at least 0.5 in. (12.7 mm) thick and have a flatness tolerance of plus or minus 0.0002 in. (0.00508 mm).

9. Test Procedure

9.1 To ensure that the applied load is uniformly distributed over the specimen end surfaces, the specimen shall be centered on the axis of the test machine.

9.1.1 Steel endplates shall be used to transfer the test loads uniformly into the stub columns (Figure 2).

9.1.2 A 0.5 in. (12.7 mm) thick layer of grout, similar to gypsum-based concrete capping compound used for fast setting, shall be placed between the stub-column endplates and the machine heads to facilitate aligning the test specimen (Figure 2).

9.2 When an axial compression load is applied to the test specimen as a result of grout expansion during curing, or if a small preload is purposely applied to ensure proper contact between the stub-column endplates and the machine heads, the load shall be treated as part of the applied test load.

9.3 The load increments applied during the test shall not exceed 10 percent of the estimated ultimate test load.

9.4 The maximum loading rate between load increments shall not exceed a corresponding applied stress rate of 3 ksi (21 MPa) of cross-sectional area per minute.

9.5 When axial shortening values are recorded, the following procedures shall be required:

(1) The change in the vertical distance between the inside surfaces of the endplates (Figure 2) shall be measured to the nearest 0.0001 in. (0.00254 mm) at each load increment for each specimen.

(2) The load increments applied during the test shall be the same for each specimen within a test unit, with a variation not to exceed one percent.

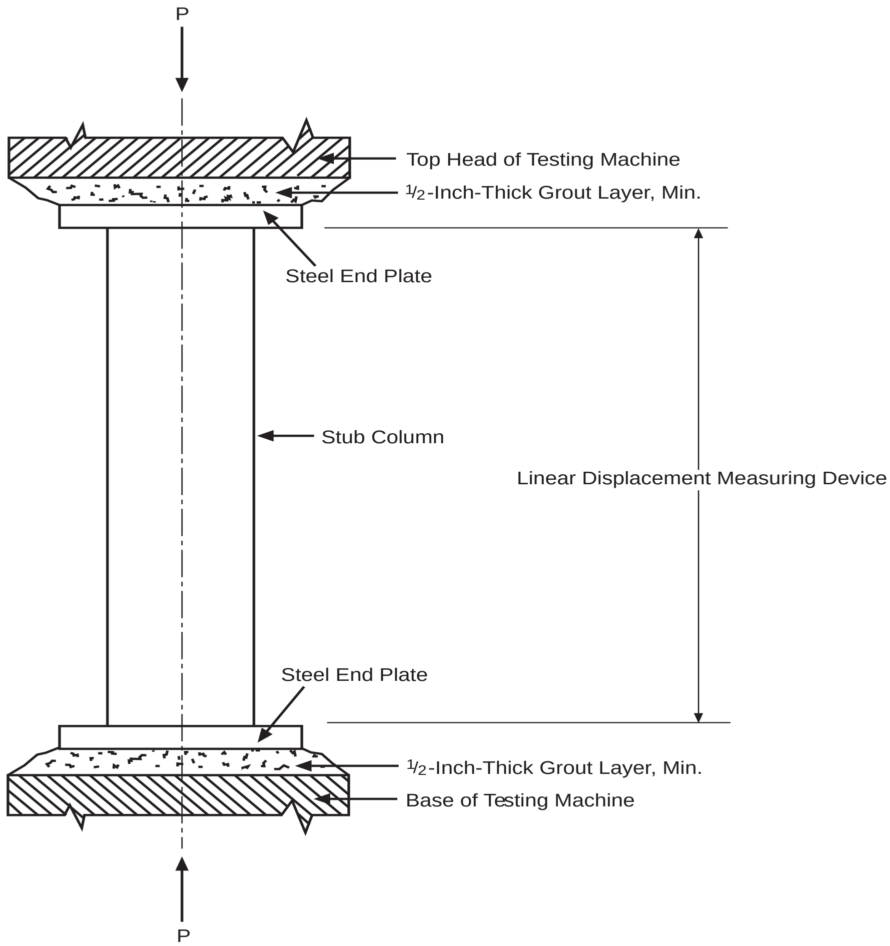


Figure 2 – Test Setup

10. Data Evaluation

10.1 For a given test unit, all individual ultimate loads, P_u, derived from the stub-column tests shall be used to calculate the average ultimate load, P_{ua}. Similarly, all individual yield stresses, F_{yi}, derived from the tensile tests of the same unit shall be used to calculate the average yield stress of the same test unit, F_{ya}.

10.2 The effective areas A_{eua}, A_{euN}, and A_e shall be calculated as specified in Sections 10.3 through 10.6; however, the final value of these effective areas shall not exceed that of the minimum gross cross-sectional area, A.

10.3 For tests in which the length of the stub column does not exceed twenty times the minimum radius of gyration of the cross section, r, the average effective area at the ultimate load, A_{eua}, for a given test unit shall be determined as follows:

$$A_{eua} = P_{ua}/F_{ya} \quad (1)$$

10.4 For tests in which the length of the stub column exceeds twenty times the minimum radius of gyration of the cross section, the average effective area at the ultimate load shall be determined by iteration of the following equations:

$$A_{eua} = A_a - \left(A_a - \frac{P_{ua}}{F_n}\right)\left(\frac{F_n}{F_{ya}}\right)^n \tag{2}$$

where A_a = average minimum area of the stub columns in the test unit, and F_n = flexural or torsional-flexural buckling stress derived from Section C4 of the *Specification* with K = 0.5 (using the average cross-sectional properties of the test unit). The exponent n shall be determined as follows:

$$n = A_{eua}/A_a \tag{3}$$

The exponent, n, shall be permitted to be obtained using the following iteration process until n is converged:

(a) Assuming an initial value for n equal to or less than 1.0,

(b) Calculate A_{eua} using Equation (2),

(c) Obtain the new n value from Equation (3).

A converged A_{eua} for one specific test unit shall be expected by repeating the above process.

10.5 The value of A_{eua} for a specific test unit shall be adjusted to A_{euN}, which is the effective cross-sectional area of a column at ultimate load with a nominal cross section of A_N and a specified minimum yield stress of F_{yN}. The adjustment shall be performed in accordance with Section 10.5.1, Section 10.5.2, or Section 10.5.3.

10.5.1 If the average area of the stub columns in the test unit, A_a, or the average base steel thickness, t_a, are different from the nominal area or thickness, respectively, the effective cross-sectional area at ultimate load shall be calculated as follows:

$$A_{euN} = A_{eua}\left(\frac{A_N}{A_a}\right) \tag{4}$$

or

$$A_{euN} = A_{eua}\left(\frac{t_N}{a_a}\right) \tag{5}$$

10.5.2 If the average yield stress of all stub columns in a test unit, F_{ya}, is different from the nominal yield stress, F_y, the effective cross-sectional area at ultimate load shall be the lower of the two values calculated as follows:

$$A_{euN} = A_N\left[1 - \left(1 - \frac{A_{eua}}{A_N}\right)\left(\frac{F_{yN}}{F_{ya}}\right)\right] \tag{6}$$

or

$$A_{euN} = A_{eua}\left(\frac{F_{ya}}{F_{yN}}\right)^{0.4} \qquad (7)$$

10.5.3 If the average area and the minimum specified yield stress are different from the nominal values of a test unit, A_{euN} derived from the equation (4) or (5) shall be used as A_{eua} in the equations (6) and (7), which will lead to an acceptable value of A_{euN}.

10.6 The effective area at any working stress level, A_e, shall be permitted to be determined as follows:

$$A_e = A_N - (A_N - A_{euN})\left(\frac{f}{F_{yN}}\right)^{n} \qquad (8)$$

10.7 For a series of sections, such as in a parameter study during which only one parameter (thickness, depth, width, yield stress, etc.) is changed, interpolations between test units, or extrapolations beyond test units, shall be permitted as described in Appendix B.

10.8 Extrapolations beyond 20 percent of the extreme parameters tested shall not be permitted.

11. Report

11.1 The report shall include a complete record of the sources and locations of all stub-column and tensile-test specimens and shall describe whether the specimens were taken from one or several columns, one or several production runs, coil stock, or other sources.

11.2 The report shall include all measurements taken for each stub-column test specimen, including (1) cross-section dimensions, (2) uncoated sheet thickness, (3) longitudinal yield stress, (4) end preparation procedure, (5) applicable material specification, and (6) test and evaluation procedure used.

11.3 The determination of the selected stub-column length shall be fully documented with appropriate calculations.

11.4 A description of the test setup - including the endplates, the grout layer used for alignment, and the instrumentation used to measure lateral displacements and axial shortening - shall be included.

11.5 The report shall include the load increments, rate of loading, and intermediate and ultimate loads for each stub column tested.

11.6 The report shall include complete calculations and results of the effective area, A_{euN}, for each test unit and calculations of A_e, if requested.

12. Precision

The criteria in Section 12.1 and Section 12.2 shall be used to judge the acceptability of the test results.

12.1 Repeatability - Individual stub-column test results shall be considered suspect if they differ by more than 10 percent from the mean value for a test unit with at least three specimens.

12.2 Reproducibility - The results of tests on stub-columns conducted at two or more laboratories shall agree within ten (10) percent when adjusted for differences in cross sectional dimensions and yield stress.

Appendix A
Use of Axial Shortening Measurements in Design

A-1 Axial shortening measurements as part of thin-walled cold-formed steel stub-column tests shall be permitted to be used as an alternative method of determining the effective area of a column, A_e, at a certain design load or stress.

A-2 The calculations by this method shall be made separately for each stub-column specimen within a test unit. This shall result in a total of j calculations as a result of a total of j load-displacement tests for each test unit.

A-3 For a given specimen the effective area at ultimate load, A_{eu}, shall be calculated from Section 10.3 or 10.4 letting $A_{eua} = A_{eu}$, $A_a = A$, $F_{ya} = F_y$, and $P_{ua} = P_u$.

A-3.1 Calculations at each load-displacement reading, i, shall be conducted in accordance with the procedure specified in this section; however, at zero load, the effective area, A_e, shall be equal to the minimum cross-sectional area, A. This provides results for the effective area at each load point:

(1) Starting with the lowest load-displacement reading, the effective area, A_i, and the assumed uniformly distributed stress f_i, shall be calculated for each reading, i, from:

$$A_{ei} = \frac{P_i D_u}{F_y D_i} \quad \text{(AA-1)}$$

and

$$f_i = \frac{F_y D_i}{D_u} \quad \text{(AA-2)}$$

where D_i and D_u are the axial shortening at loads P_i and P_u, respectively.

(2) If A_{ei} calculated is greater than A, A_{ei} shall be set equal to A.

(3) If A_{ei} calculated is less than A, A_{ei} shall be as calculated, and f_o, the stress above which the section is not fully effective, shall be set equal to f_{i-1}, as calculated for the previous load-displacement reading.

A.3.2 For specimens within a test unit, the lowest A_{ei} values shall be used for further evaluations.

A-4 For any load that causes a stress f higher than f_o, an exponential equation shall be permitted to be developed as follows:

$$A_e = A\left[1-\left(1-\frac{A_{eu}}{A}\right)\left(\frac{f-f_o}{F_y-f_o}\right)^b\right] \quad \text{(AA-3)}$$

where

$$b = \frac{\sum_{i=1}^{j}(X_i)(Y_i) - (a)\sum_{i=1}^{j}(X_i)}{\sum_{i=1}^{j}(X_i)^2} \tag{AA-4}$$

and

$$X_i = \ln\left(\frac{f_i - f_o}{F_y - f_o}\right) \tag{AA-5}$$

$$Y_i = \ln\left(1 - \frac{A_{ei}}{A}\right) \tag{AA-6}$$

$$a = \ln\left(1 - \frac{A_{eu}}{A}\right) \tag{AA-7}$$

and ln designates the natural logarithm.

A-5 If the effective areas for a section with specified dimensions and minimum yield stress are desired, which are different from the tested specimens, the A_{eu} and A_{ei} values calculated under Section A-3 shall be normalized to the specified parameters in accordance with Section 10.5 before the curve-fitting procedure of Section A-4 is employed. The variables A, A_{eu} and F_y shall be changed to A_N, A_{euN} and F_{yN}.

A-6 All calculations pertaining to this procedure shall be included in the report, as discussed in Section 11.

Appendix B
Parametric Studies

B-1 For parametric studies intended to develop the effective area for a series of sections with the same basic cross section (either C, U, H, or any other shape) and the same hole pattern, but with one or more changing parameters, the required number of test units shall be permitted to be less than the sum of all sections with different geometries and yield stresses.

B-1.1 For a series of sections with three different values for one parameter only (dimension or nominal yield stress), at least two test units shall be chosen to include the minimum and the maximum value of the changing parameter. For the third value, A_{eu} shall be permitted to be interpolated in accordance with Section B-2.

B-1.2 If more than three different values for one parameter are included in a series of sections, additional units with intermediate values shall be tested such that the ratio of the changing values in adjacent units is not greater than 1.5 or be less than 0.67. For intermediate values of the changing parameter, A_{eu} shall be permitted to be interpolated according to Section B-2.

B-1.3 For a series of sections with the same basic cross section that includes different values for several parameters (dimensions and/or yield stress), an appropriate factorial of test units shall be established by the responsible professional engineer in accordance with the guidelines for changes in an individual parameter, and in compliance with responsible code authorities. Interpolations and extrapolations shall be permitted to be made as mutually agreeable, following the general guidelines set forth in Section B-2 for changes of one parameter only.

B-1.4 For a section that falls outside a series of tested members with the same basic cross section, A_{eu} shall be permitted to be extrapolated provided the changing parameter does not exceed a value of 20 percent below or above the respective minimum or maximum values tested in the series.

B-2 Interpolations and extrapolations shall be permitted as part of a parametric study, and as defined under B-1.

B-2.1 For a section with a thickness different from the thicknesses tested, but with identical overall nominal cross-sectional dimensions and minimum specified yield stress, A_{eu} for a thickness t and an area A shall be permitted to be calculated provided t does not exceed the limits described under Section B-1.2 and B-1.4. Under these conditions, A_{eu} shall be permitted to be determined by interpolation or extrapolation from the results of the nearest two test units with thicknesses t_1 and t_2, respectively.

$$A_{eu} = A\left[\frac{A_{eu1}}{A_1} + \left(\frac{A_{eu2}}{A_2} - \frac{A_{eu1}}{A_1}\right)\left(\frac{t_1 - t}{t_1 - t_2}\right)\right] \quad \text{(AB-1)}$$

where A_1 and A_2 are the minimum gross cross-sectional areas, and A_{ue1} and A_{eu2} are the nominal effective cross-sectional areas for Test Units 1 and 2, respectively.

B-2.2 For a section with a yield stress different from the yield stresses tested, but with identical cross-sectional dimensions, A_{eu} for a yield stress F_y shall be permitted to be calculated provided F_y does not exceed the limits described under Section B-1.2 and B-1.4. Under these conditions, A_{eu} shall be permitted to be determined by interpolation or extrapolation from the results of the nearest two test units with yield stresses F_{y1} and F_{y2}, and with effective areas A_{eu1} and A_{eu2}, respectively.

$$A_{eu} = A\left[\frac{A_{eu1}}{A_1} + \left(\frac{A_{eu2}}{A_2} - \frac{A_{eu1}}{A_1}\right)\left(\frac{F_{y1} - F_y}{F_{y1} - F_{y2}}\right)\right] \qquad \text{(AB-2)}$$

Commentary on AISI S902-08

Stub-Column Test Method for Effective Area of Cold-Formed Steel Columns

The effective area is used to determine the nominal axial strength of cold-formed column sections in accordance with AISI S100, *North American Specification for the Design of Cold-Formed Steel Structural Members*, hereafter referred as AISI S100.

The effective area is a variable section property of columns. It reflects the effects of local buckling in relatively thin area elements caused by axial stresses, or loads. When the axial load is zero, the effective area is equal to the gross cross-sectional area; however, when an axial load is applied, the effective area may be less than the gross area. In such a case, the effective area will reduce with increasing load.

Local buckling reduces the axial load-carrying capacity that would otherwise be limited only by general yielding or overall column buckling. The amount of the reduction depends on the width-to-thickness ratio of the flat elements of the column cross section, the yield stress of the steel sheet from which the column is formed, and the size and frequency of holes or hole patterns, if present.

Research information can be found in Reference 1 on the determination of the effective cross-sectional area of cold-formed steel columns and the consideration of the effects of local buckling and residual stresses on solid or perforated columns that have holes (or hole patterns) in the flat and/or curved elements of the cross section.

The effective area, A_{euN}, of a cold-formed column section at ultimate load and the load- or stress-dependent effective area, A_e are used in the AISI S100 to determine the ultimate and less-than-ultimate column strengths. The ultimate column strength, P_N, is the product of the minimum specified yield stress, F_{yN}, or the buckling stress F_n, and the corresponding effective cross-sectional area at that stress, A_{euN}. At an applied column strength of P less than P_N, the corresponding effective cross-sectional area is A_e.

An inherent assumption of the test method is that true stub-column behavior (which considers local buckling effects only) is achieved when overall column-buckling effects are eliminated. For this condition the ultimate test load on a stub-column, P_u, equals the product of the effective cross-sectional area at ultimate load, A_{eu}, times the stress that causes local buckling, or times the yield stress of the virgin steel sheet. In case overall buckling cannot be avoided because of geometrical constraints, the critical column-buckling stress must be used.

The following two approaches are provided in the Standard for determining the effective area, A_e:

(1) The basic, more simple and conservative method:

This method is embodied in the main part of this document and is based on the measured test loads of stub-columns and their measured and tested physical and mechanical properties.

(2) An alternative and less conservative method provided in Appendix A. This method is based on the shortening of stub-columns which occurs during testing. Also, this evaluation method requires more calculations. The results of this method lead to more accurate results for Ae, and to higher allowable axial loads at lower-than-ultimate stress levels. The evaluation procedure for this method is described in Appendix A.

References

1. T. Pekoz, "Development of a Unified Approach to the Design of Cold-Formed Steel Members", Committee of Sheet Steel Producers, American Iron and Steel Institute, 1140 Connecticut Avenue, Washington, DC 20036, 1986.

AISI S903-08

Standard Methods for Determination of Uniform and Local Ductility

1. Scope

This method covers the determination of uniform and local ductility from a tension test.

2. Referenced Documents

The following documents or portions thereof are referenced within this Standard and shall be considered as part of the requirements of this document.

a. American Iron and Steel Institute (AISI), Washington, DC:

S100-07, *North American Specification for the Design of Cold-Formed Steel Structural Members*

b. ASTM International (ASTM), West Conshohocken, PA:

A370-07b, Standard Test Methods and Definitions for Mechanical Testing of Steel Products

3. Symbols

e_1 = Linear elongation, in., in 1-in (25.4 mm) gage length

e_3 = Linear elongation, in., in 3-in. (76.2 mm) gage length

e_{3e} = Linear elongation, in., in 2-in. (50.8 mm) gage length not containing 1-in. (25.4 mm) length of fractured portion

e_u = Linear elongation, in., at ultimate load in standard tension coupon test

ε_3 = Percent elongation in 3-in. (76.2 mm) gage length

ε_{3e} = Percent elongation in 2-in. (50.8 mm) gage length not containing 1-in. (25.4 mm) length of fractured portion

ε_f = Percent elongation at fracture in 2-in. (50.8 mm) gage length of standard tension coupon

ε_u = Percent elongation at ultimate load in standard tension coupon test

$\varepsilon_{uniform}$ = Uniform percent elongation

ε_{local} = Local percent elongation in 1/2 in. (12.7 mm) gage length

$\varepsilon_{1/2}$ = Percent elongation in 1/2 in. (12.7 mm) gage length

4. Test Procedure

4.1 A tension coupon shall be prepared in accordance with ASTM A370 except that the central length of 1/2 in. (12.7 mm) uniform width of the coupon shall be at least $3^1/_2$ in. (88.9 mm) long.

4.2 Gage lines shall be scribed at 1/2-in. (12.7 mm) intervals along the entire length of the coupon.

4.3 After completion of the coupon test, the following two permanent plastic deformations shall be measured: (a) the linear elongation in a 3-in. (76.2 mm) gage length, e_3, such that the fractured portion is included (preferably near the middle third of this 3-in. (76.2 mm) gage length); and (b) the linear elongation in a 1-in. (25.4 mm) gage length, e_1, containing the fracture.

4.4 The linear elongation, e_{3e}, shall be calculated in accordance with Equation (1), where e_{3e} = linear elongation in a 2-in. (50.8 mm) gage length not containing the 1-in. (25.4 mm) length of the fractured portion.

$$e_{3e} = e_3 - e_1 \qquad (1)$$

4.5 From the two preceding elongation measurements, e_3 and e_{3e}, calculate the percentage elongations as follows:

$$\varepsilon_3 = (e_3/3.00) \times 100 \quad \text{in inches} \qquad (2a)$$

$$= (e_3/76.2) \times 100 \quad \text{in millimeters} \qquad (2b)$$

$$\varepsilon_{3e} = (e_{3e}/2.00) \times 100 \quad \text{in inches} \qquad (3a)$$

$$= (e_{3e}/50.8) \times 100 \quad \text{in millimeters} \qquad (3a)$$

From these percentage elongations, the uniform and local ductility parameters shall be obtained in accordance with Sections 4.6 and 4.7.

4.6 Since the fractured portion which includes local elongation is eliminated from ε_{3e}, it shall be a measure of the uniform ductility of the material. Thus

$$\varepsilon_{uniform} = \varepsilon_{3e} \qquad (4)$$

4.7 The local elongation shall be determined over a small length which includes the fractured portion. For simplicity, this length shall be permitted to be assumed to be 1/2 in. (12.7 mm) which is large enough to include the necked portion of most thicknesses and type of sheet steels used, and is small enough to give valid comparison for different types of steels. Thus

$$\varepsilon_{local} = \varepsilon_{1/2} = 6\,(\varepsilon_3 - \varepsilon_{3e}) + \varepsilon_{3e} \qquad (5)$$

in which constant 6 is the multiplication factor which converts the local elongation ($\varepsilon_3 - \varepsilon_{3e}$) measured in 3 in. (76.2 mm) to local elongation in 1/2 in. (12.7 mm) gage length.

5. Alternative Test Procedure

The alternative test procedure provided in this section shall be permitted to be used in lieu of the test procedure provided in Section 4 for determining the local elongation.

5.1 A standard tension coupon shall be prepared in accordance with ASTM A370 with a standard 2-in. (50.8 mm) gage length.

5.2 The strain at the tensile strength, i.e., percentage strain ε_u at the peak of the stress-strain curve, shall be measured. The percentage elongation, ε_u, at ultimate load shall then be calculated as

$$\varepsilon_u=(e_u/2.00) \times 100 \quad \text{in inches} \qquad (6a)$$

$$=(e_u/50.8) \times 100 \quad \text{in millimeters} \qquad (6b)$$

5.3 To obtain a measure of the local ductility, percentage strain at fracture ε_f, also in a 2-in. (50.8 mm) gage length, shall be measured. Equation (7) shall then be used to convert ($\varepsilon_f - \varepsilon_u$) into the percentage elongation in a 1/2 in. (12.7 mm) gage length:

$$\varepsilon_{local} = \varepsilon_{1/2} = \varepsilon_u + 4\,(\varepsilon_f - \varepsilon_u) \qquad (7)$$

in which 4 is the multiplication factor to convert a 2-in. (50.8 mm) gage length local elongation to a 1/2 in. (12.7 mm) gage length.

Commentary on AISI S903-08

Standard Methods for Determination of Uniform and Local Ductility

This Standard is developed to determine uniform and local ductility from a tension test. The test method provides an alternative method of determining if a steel has adequate ductility as defined in the AISI S100, *North American Specification for the Design of Cold-Formed Steel Specification*. This test method is based on the method suggested by Dhalla and Winter (1974).

In the Standard, two test procedures are provided. In the first test procedure provided in Standard Section 4, linear elongation in a 3-in. (76.2 mm) gage length, e_3, and linear elongation in a 2-in. (50.8 mm) gage length, e_1, are measured. Both elongations contain the fractured portion. The difference of e_3 and e_1 gives the linear elongation, e_{3e}, in a 2-in. (50.8 mm) gage length not containing the 1-in. (25.4 mm) length of the fracture. ε_{3e} is used to measure the uniform ductility. The local elongation, which should include the fractured portion, is determined using the Standard Equation (5)

In the alternative method provided in Standard Section 5, the strain at the tensile strength, i.e., percentage strain ε_u at the peak of the stress-strain curve, is a measure of uniform ductility, because up to this strain no necking or local elongation has taken place. Therefore, to obtain the uniform ductility the stress-strain curve is plotted at least up to the maximum load or the linear elongation, e_u, at maximum load is measured directly, so that ε_u is obtained by Standard Equation (6a) or (6b).

To obtain a measure of the local ductility, it is necessary to measure the percentage strain at fracture ε_f, also in a 2-in. (50.8 mm) gage length. However, the strain which occurs after the maximum load has been passed (descending branch) is the necking strain, and is localized at the eventual fracture zone, thus (ε_f - ε_u) is the local percentage elongation referred to in a 2-in. (50.4 mm) gage length. The Standard Equation (7) converts this (ε_f - ε_u) into the percentage elongation in a 1/2 in. (12.7 mm) gage length.

Reference

Dhalla, A. K. and G. Winter (1974), "Steel Ductility Measurements," Journal of Structural Division, Proceedings ASCE, Vol. 100, No. ST2, February 1974.

AISI S904-08

Standard Test Methods for Determining the Tensile and Shear Strength of Screws

1. Scope

1.1 These performance test methods establish procedures for conducting tests to determine the tensile and shear strength of carbon steel screws. The screws may be thread-forming or thread-cutting, with or without a self-drilling point, and with or without washers. The intended application for these screws is to connect cold-formed sheet steel material.

1.2 Two test methods are included in this Standard:

Tensile Tests. This test is intended to determine the ability of a screw to withstand a load when applied along the axis of the screw.

Single Shear Test. This test is intended to determine the ability of a screw to withstand a load applied transversely to the axis of the screw.

1.3 These standard test methods describe mechanical tests for determining the following properties:

	Section
Tensile Strength	4.1
Single Shear Strength	4.2

1.4 These standards do not intend to address all of the safety concerns, if any, associated with their use. It is the responsibility of the user of these standards to establish appropriate safety and health practices, and determine the applicability of regulatory limitations prior to use.

2. Referenced Documents

The following documents or portions thereof are referenced within this Standard and shall be considered as part of the requirements of this document.

a. American Iron and Steel Institute (AISI), Washington, DC:

S100-07, *North American Specification for the Design of Cold-Formed Steel Structural Members*

b. ASTM International (ASTM), West Conshohocken, PA:

A370-07b, Standard Test Methods and Definitions for Mechanical Testing of Steel Products

E6-07b, Standard Terminology Relating to Methods of Mechanical Testing

F606-07, Standard Test Methods for Determining the Mechanical Properties of Externally and Internally Threaded Fasteners, Washers, and Rivets

IEEE/ASTM-SI-10-02, American National Standard for Use of the International System of Units (SI): The Modern Metric System

3. Symbols

w = Test specimen width
L = Test specimen length excluding gripped end
e = Distance from center of the hole to the end of the specimen
p = Spacing of bolt holes.

4. Test Methods

A test series shall be conducted for each screw material grade, head type, thread series and nominal diameter.

4.1 Tensile Tests:

4.1.1 The screw shall be tested in a holder with the load axially applied between the head and a suitable fixture, which shall have sufficient thread engagement to develop the full strength of the screw. See Figure 1 for a standard tensile test setup.

User Note:
Threads may be clamped directly by jaws of testing machine if screw shank is not crushed in so doing.

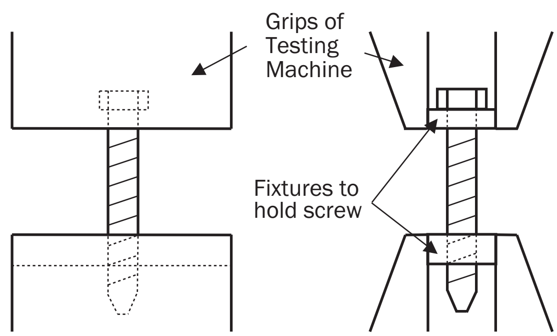


Figure 1 – Standard Tensile Test

4.1.2 The speed of testing, as determined by the rate of separation of the testing machine heads, shall be limited to the greater of 0.1 in. (2.5 mm) per minute or the separation rate caused by a loading rate of 500 pounds (2 kN) per minute.

4.1.3 The maximum load applied to the specimen, coincident with or prior to screw failure, shall be recorded as the tensile strength of the screw.

4.2 Single Shear Test

4.2.1 The specimen shall be tested using steel plates or shapes of sufficient thickness to preclude bearing failure and ensure failure through the fully-threaded section. The shear plates or shapes shall create a single-lap joint connected with one or two fasteners. If two fasteners are used, the total shear strength of the connection shall be divided by two to determine the shear strength for one screw. Geometrical proportions of the test specimen shall be as suggested in Table 1, with reference to Figures 2 and 3. The test fixture shall provide for central loading across the lap joint. When the machine grips are adjustable, or when the thickness of either plate is less than 1/16 in. (2 mm), packing shims shall not be required for central loading.

Table 1 - Geometrical Proportions of Specimen

Screw Diameter, d in. (mm)	w in. (mm)	L in. (mm)	e in. (mm)	p in. (mm)
≤ 0.250 (6.5)	2 (50)	Min. 10 (250)	1 (25)	2 (50)
> 0.250 (6.5)	8d	Min. 10 (250)	3d > 1 (25)	3d > 2 (50)

4.2.2 The test specimen shall be permitted to be assembled in a shear fixture or threaded into two flat sheets. The test specimen shall be mounted in a tensile-testing machine capable of applying load at a controllable rate. The grips shall be self-aligning and care shall be taken when mounting the specimen to assure that the load will be transmitted in a straight line transversely through the test screw(s). Load shall be applied and continued until failure of the screw(s). Speed of testing, as determined by the rate of separation of the testing machine heads, shall be limited to the greater of 0.1 in. (2.5 mm) per minute or the separation rate caused by a loading rate of 500 pounds (2 kN) per minute.

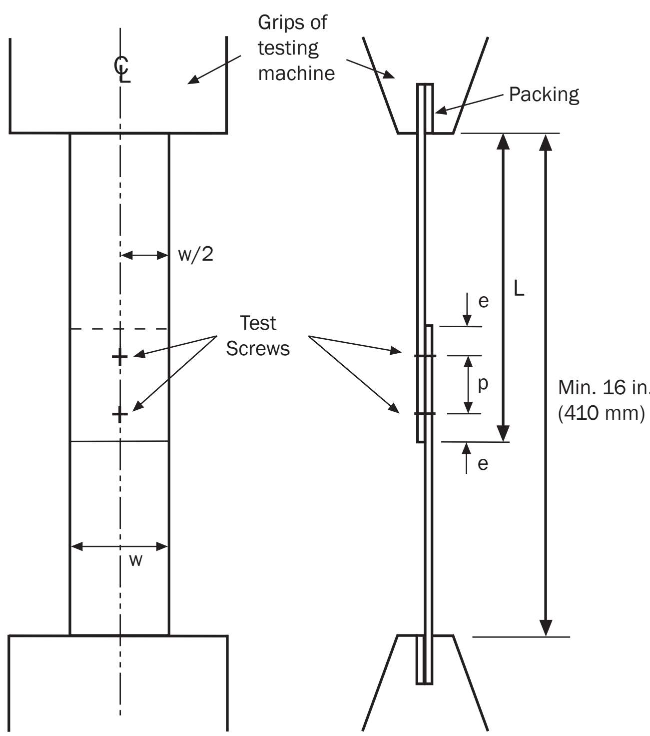


Figure 2 – Standard Lap-Joint Test – 2 Screws

4.2.3 The maximum load applied to the specimen, coincident with or prior to screw failure, shall be recorded as the shear strength of the screw.

5. Report

5.1 The objectives and purposes of the test series shall be stated at the outset of the report so that the necessary test results such as the maximum load per fastener and the mode of failure are identified.

5.2 The type of tests, the testing organization, the supervising engineer, and the dates on which the tests were conducted shall be included in the documentation.

5.3 The test specimen shall be fully documented, including:

(a) The measured dimensions and identification data of each specimen:

- thread O.D.
- thread I.D.
- threads per unit length
- head dimensions
- screw length
- manufacturer
- designation or type
- unthreaded length or imperfect threads below head
- grade of material
- drill-point diameter and length of flutes for self-drilling screws
- any other distinguishing characteristics

(b) The details of fastener installation including pre-drilling, diameter of the pilot drill if used, tightening torque, and any unique tools used in the installation, and

(c) Identification of the washers or washer-head data including diameter, thickness, material, and data on the sealant if present.

5.4 The test set-up shall be fully described including the type of testing machine, the specimen end grips or supports.

5.5 The test procedure shall be fully documented including the rate of loading.

5.6 In accordance with the test objectives stated by the responsible engineer, the report shall include a complete documentation of all applicable test results for each specimen such as the maximum load and the mode of failure. The report shall also include the necessary calculations for the screw design strength and safety factors/resistance factors based on the requirements specified in Section F1 of AISI S100.

AISI S905-08

Test Methods for Mechanically Fastened Cold-Formed Steel Connections

1. Scope

1.1 These performance test methods cover the determination of the strength and deformation of mechanically fastened connections for cold-formed steel building components. Connections in which the fasteners are stressed in shear (loads applied perpendicular to the shank of the fastener) and those in which the fasteners are stressed in tension (loads applied parallel to the shank of the fastener) are included. The objective is to evaluate field connections using standard test specimens and fixtures as provided in Section 8.

1.2 For circumstances in which geometric eccentricities exclude the use of the standard tests, alternative tests are permitted. Requirements for both standard tests and alternative tests are provided.

1.3 Connections of thin components (such as exterior building sheet steel) to relatively thick components (such as structural frame supports) are considered as well as connections between thin components.

1.4 Mechanical fasteners include rivets, screws, bolts, and power-actuated fasteners.

1.5 The mode of failure incurred during a test is critical and must be reported.

1.6 The subject test methods are concerned with determining the strength characteristics of connections in which the fasteners do not fail. It is intended that sufficient fastener strength will be provided to cause failure in the connected members.

1.7 The test methods are suitable for hydraulic or screw operated testing machines using force or displacement control.

2. Referenced Documents

The following documents or portions thereof are referenced within this standard and shall be considered as part of the requirements of this document.

a. American Iron and Steel Institute (AISI), 1140 Connecticut Avenue, NW, Washington, DC 20036:

S100-07, *North American Specification for the Design of Cold-Formed Steel Structural Members*, 2007 Edition

b. ASTM International, Inc, 100 Barr Harbor Drive, West Conshohocken, Pennsylvania 19428-2959:

A370-07b, Standard Test Methods and Definitions for Mechanical Testing of Steel Products

E6-07b, Standard Terminology Relating to Methods of Mechanical Testing

E1190-95(2007), Standard Test Methods for Strength of Power-Actuated Fasteners Installed in Structural Members

c. American Society of Mechanical Engineers (ASME), Three Park Avenue, New York, New York 10016-5990:

ANSI/ASME B18.6.4-1999, Thread Forming and Thread Cutting Tapping Screws and Metallic Drive Screws, Inch Series

3. Terminology

Where the following terms appear in this Standard they shall have the meaning as defined herein. Terms not defined in Section 3 of this Standard, AISI S100 or ASTM E6 shall have the ordinary accepted meaning for the context for which they are intended.

Characteristic connection strength per fastener. A statistically adjusted value for the mean maximum load per fastener measured for the test unit.

Diaphragm. Roof, floor or other membrane system that transfers in-plane forces to the lateral force resisting system.

Gross distortion. A failure mode where a steel sheet undergoes very large permanent deformation prior to failure at the fasteners.

Load-deformation curve per fastener. The load-deformation curve for the test connection with the load values divided by the number of fasteners.

Maximum load per connection. The maximum load recorded during a test.

Maximum load per fastener. The maximum connection load divided by the number of fasteners in the connection.

(Fastener) Pull-out. A failure mode where fasteners are pulled free from the support.

Pull over (or pull through). A failure mode where a steel sheet is pulled over the fastener head (i.e. the fastener head is pulled through the sheet steel).

Stitch Connector. A fastener connecting adjacent steel sheets to each other, but not connecting to the heavier frame or structural members. Also referred to as a sidelap connector.

Structural Connector. A fastener connecting one or more steel sheets to a heavier frame or structural member.

4. Symbols

The symbols used in this Standard shall be defined as follows:

A = Elongation in a tension test
a = Width of the end supports in the alternative tension tests
a_s = Shear deformation or slip
a_t = Deformation in a tension test
b = Width of the troughs or flats in a profiled sheet
u_s = Shear flexibility per fastener
u_t = Tensile (uplift) flexibility per fastener
d = Nominal diameter of a fastener
e_1 = End distance of a fastener in a standard test specimen
e_2 = Fastener edge distance in an alternative test specimen
h = Height of the stiffening ribs in a profiled sheet
L_s = Free strap length in a standard shear test specimen
L_a = Test span for an alternative tension test specimen
ℓ_g = Extensometer gage length in a shear test
M_u = Maximum bending moment per stiffening rib

N = Number of fasteners in lap-joint connection as shown in Figures 1 and 2
n = Number of valid tests in a test unit
P = Estimated maximum connection load per fastener
P_K = Characteristic connection strength per fastener
P_m = Mean connection strength per fastener
P_u = Maximum connection strength
Q_f = Structural connector strength for sheet to base steel in diaphragm
Q_s = Stitch connector strength for sheet to sheet steel in diaphragm
S_f = Structural connector flexibility for sheet to base steel in diaphragm
S_s = Stitch connector flexibility for sheet to sheet steel in diaphragm
P_{nov} = Structural connector pullover strength for sheet to base steel
F_y = Yield stress
F_u = Tensile strength
s = Standard deviation
p = Fastener spacing or pitch
t = Base metal sheet thickness
w = Width of the shear test specimen
Δ_g = Total elongation of extensometer gage length, ℓ_g, at 40 percent of P_u in a standard or alternative shear test

5. Units of Symbols and Terms

Any compatible system of measurement units shall be permitted to be used in this Standard, except where explicitly stated otherwise. The unit systems considered in this Standard shall include U.S. customary units (force in kips and length in inches) and SI units (force in Newtons and length in millimeters) in accordance with IEEE/ASTM-SI-10. Quantities for SI units are provided in parenthesis.

6. Apparatus

6.1 The rate of loading shall be controlled, constant loads shall be maintained, and the applied load shall be measured accurately to within ± 2 percent.

> **User Note:**
> It is recommended that any testing machine used complies with the requirements of ASTM E4-07, Standard Practices for Force Verification of Testing Machine.

6.2 The test specimen support fixtures and the testing machine grips shall have the capability of maintaining a constant loading direction throughout the test.

6.3 The devices used to measure deformation shall provide an accuracy of ±0.001 in. (0.02 mm) for shear tests and ± 0.002 in. (0.05 mm) for tension tests.

6.4 The devices used to measure the dimensions of the test specimens shall be accurate to within ±0.0005 in. (0.01 mm) for sheet base metal thickness, ±0.05 in. (1 mm) for sheet profile dimensions, and ±0.005 in. (0.1 mm) for fastener dimensions.

7. Test Unit

7.1 For connections that include a single cross-section, a single nominal sheet thickness, and a single nominal tensile strength for the critical connection components, a minimum of three (3) specimens shall be tested. Three (3) additional tests shall be required if any single test yields a maximum load that differs from the mean maximum load by more than 15 percent.

7.2 For evaluations that include one connection cross-section with several nominal values for the thickness or tensile strength of the critical connection components, at least three (3) tests shall be required for each (thickness and /or strength) value. The differences necessary to define distinct nominal values for sheet thickness and tensile strength shall be at least 0.005 in. (approximately 0.1 mm) and 20 ksi (approximately 30 N/mm^2), respectively.

7.3 Three (3) sheet-type tension test coupons shall be tested for each thickness and strength of steel sheet used in the fabrication of the test specimens. The tension test coupon shall be taken from a flat undamaged area of the sheet component. When the sheet component is corrugated or profiled, the tension test coupon shall be oriented parallel to the corrugations or ribs. The sheet tension tests shall be conducted in accordance with ASTM A370 and the yield stress, tensile strength, and percent elongation at fracture shall be measured. The average of the three (3) respective test values shall be regarded as the yield stress, tensile strength, and elongation.

8. Test Specimens and Fixtures

8.1 General

8.1.1 Standard shear and tension tests shall be used whenever possible. Alternative shear and tension tests shall be permitted only when the standard tests are unsuitable for evaluating the connection properties under consideration. Standard or alternative test specimens shall be permitted to be used to conduct shear tests on specimens with a single or multiple fasteners. When such tests are conducted to study the influence of end distance or edge distance, the dimensions given in Tables 1 and 2 shall be permitted to be changed as required.

8.1.2 The dimensions of the test specimen components shall be measured to an accuracy of ± 0.0005 in. (0.01 mm) for sheet base metal thicknesses, ± 0.05 in. (1 mm) for sheet profile dimensions, and ± 0.005 in. (0.1 mm) for fastener dimensions.

8.1.3 Fasteners shall be placed within ± 0.05 in. (1 mm) of their specified location and affixed in accordance with the manufacturer's recommendations or the site practice. Special note shall be taken of the following, if applicable: (1) the diameter of predrilled holes, (2) the torque and depth control for threaded fasteners, and (3) the installation tools and cartridge types used for power-actuated fasteners.

8.2 Lap-Joint Shear Tests

8.2.1 Shear test deformations shall be obtained from extensometer readings across the lap joint to the accuracy as specified in Section 6.3.

8.2.2 The standard shear test specimen shall be a single-lap joint using two flat steel components connected with 2 fasteners. See Figure 1(a). The test specimen setup shown in Figure 1(b) shall also be permitted if the sheet steel components are connected to a connection plate or base steel in a field lap-joint connection. Geometrical proportions of the specimen shall be as given in Table 1. For tests investigating edge failure, or other special conditions, dimensions e_1, p, and w shall be permitted to be modified.

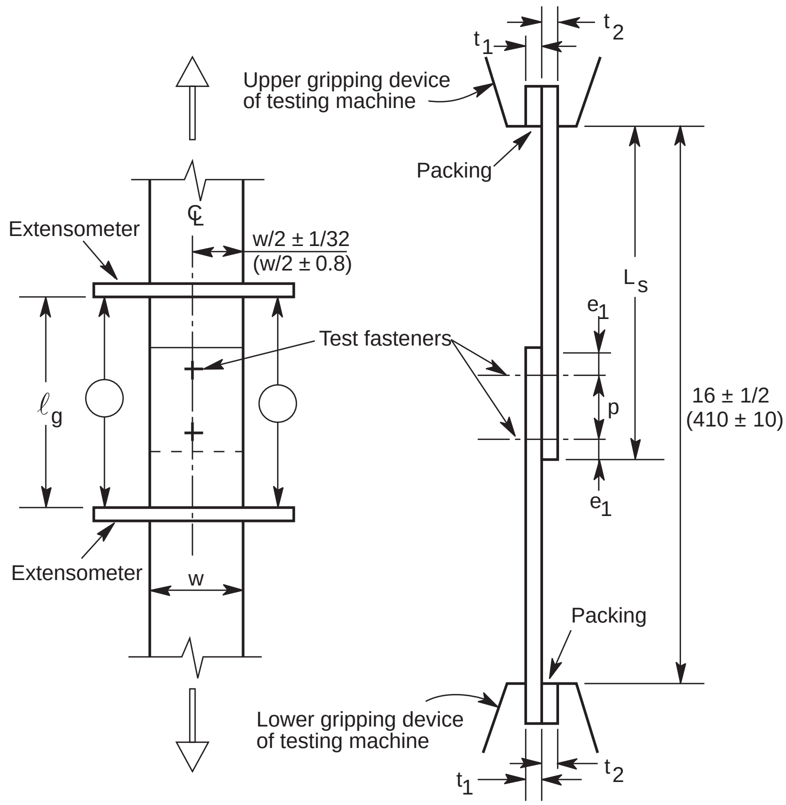


Figure 1 – Standard Lap-Joint Shear Test, units – in. (mm)

8.2.2.1 The test fixture for the standard shear test shall provide for central loading across the lap joint. See Figures 1(a) and 1(b). When the machine grips are adjustable or when the thickness of either strap is less than 1/16 in. (approximately 2 mm), packing shims shall not be required for central loading.

8.2.2.2 The extensometer gage length, ℓ_g in Table 1, shall be short enough to eliminate the influence of stretch in the specimen straps on the extensometer readings.

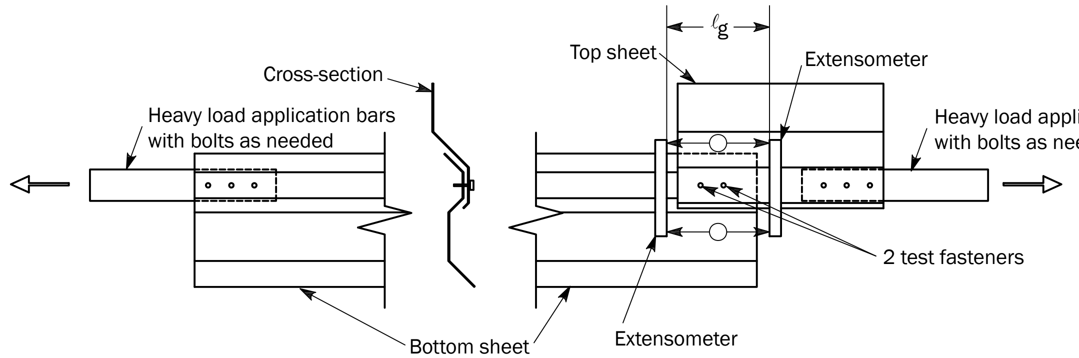
8.2.2.3 Test setups as shown in Figures 1(a) and 1(b) shall be acceptable for determination of strength and flexibility of structural and stitch sidelap connectors used in diaphragms when those connectors are in the bottom trough and not at elevated sidelaps. The edge dimension (w/2) perpendicular to the line of force shall be no greater than 2d when this is representative of the product design. Otherwise Table 1 shall apply.

8.2.3 The alternative shear test specimen setup 1 shall be a simulated diaphragm-action connection with central loading applied in the plane in which the overlapping elements are joined by the fasteners. See Figure 2(a). This specimen shall be used to determine the strength and flexibility of the diaphragm connection. Geometric proportions of the specimen shall be as given in Table 2, unless the field proportions of a diaphragm are to be tested.

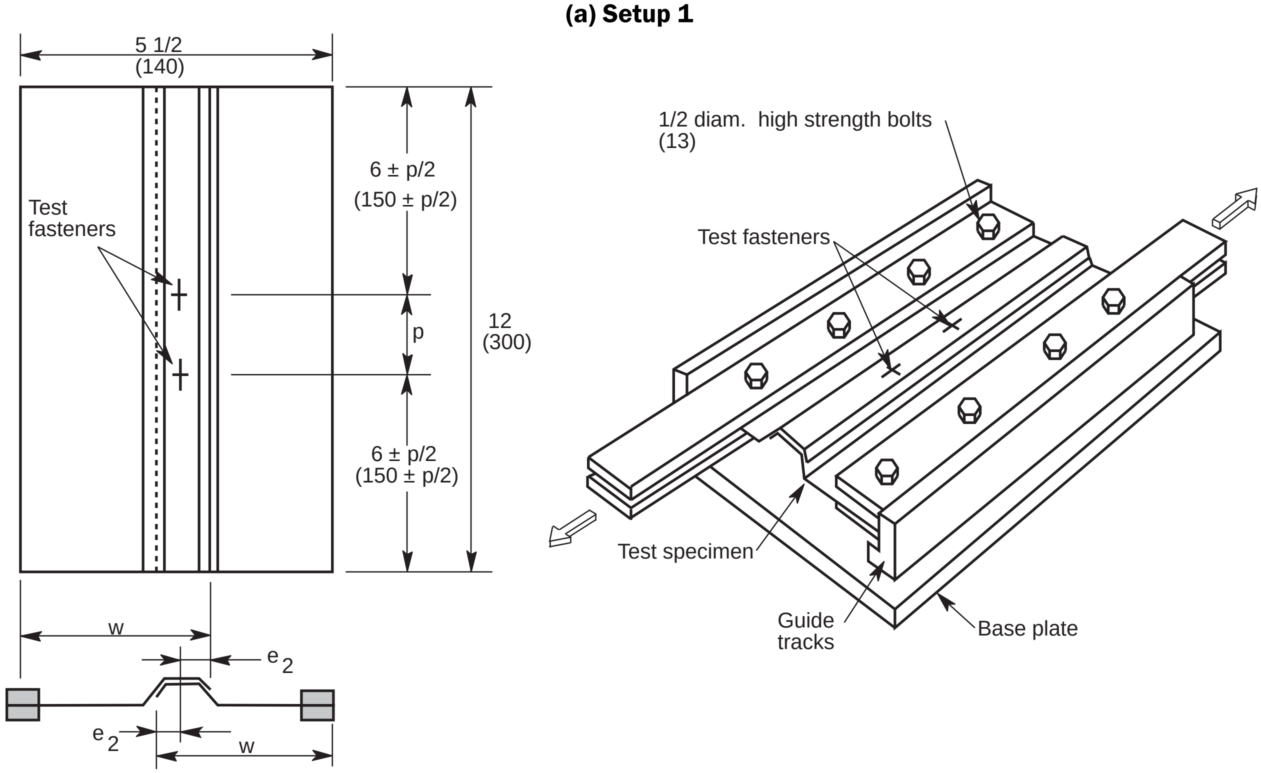
Table 1
Geometrical Proportions for Standard Lap-Joint Connection Tests

Fastener Diameter d, in. (mm)	Specimen Dimensions in., (mm)				
	w	L_s	e_1	p	ℓ_g
≤ 1/4 (6.5)	2-3/8 (60)	10-1/4 (260)	1-3/16 (30)	2-3/8 (60)	5-7/8 (150)
>1/4 (6.5)	10d (10d)	8+10d (200+10d)	5d (5d)	10d (10d)	1-3/16+20d (30+20d)
Tolerance	+1/16	+3/16	+1/32	+1/32	+3/16
	(+2)	(+5)	(+1)	(+1)	(+5)

8.2.4 The alternative shear-test specimen setup 2 shall be a simulated diaphragm-action connection with central loading applied in a plane different from that in which the overlapping elements are joined by the fasteners. See Figure 2(b). This specimen shall be used to determine the strength and flexibility of the diaphragm connection, especially for crest-connected overlapping elements of diaphragms. Geometric proportions of the specimen shall be as given in Table 2, unless the proportions of a diaphragm are to be tested. Guides shall be machined and polished, then greased.



(a) Setup 1



(b) Setup 2

Figure 2 - Alternative Lap-Joint Shear Test, units - in. (mm)

User Note:

Extensometer has not been shown in Figure 2 (b), Setup 2, for the clarity of the presentation. One end of the extensometer should be mounted on the top sheet and the other end mounted on the bottom sheet of the test specimen to be consistent with the setup as illustrated on Figure 2(a). Mounting should not resist shear and slip at the connection faying surface.

Table 2
Geometrical Proportions for Alternative Shear Tests

Fastener Diameter d, in. (mm)	Specimen Dimensions, in. (mm)			
	w	e_2	p	ℓ_g
≤ 1/4 (6.5)	3-5/8 (92.5)	3/8 (10)	2-3/8 (60)	5-7/8 (150)
> 1/4 (6.5)	3-1/4 + e_2 (82.5 + e_2)	1.5d (1.5d)	10d (10d)	1-3/16 + 20d (30 + 20d)
Tolerance	+ 1/16 (+2)	+1/32 (+1)	+1/32 (+1)	+3/16 (+5)

8.2.4.1 The alternative test fixture shall be designed such that the base plate is securely attached to a level foundation beam (or slab) and loaded in a horizontal plane using a hydraulic ram or an equivalent system. See Figure 2(b). Lubricating material shall be used to reduce the friction between guide and grips.

8.2.5 Structural and stitch sidelap connector flexibility shall be determined by measuring the displacement of the connection corresponding to 40% of the ultimate lap-joint shear capacity on the load-displacement curve ($0.4P_u$). Tests shall be conducted with minimum and maximum base steel thickness of the supporting member in accordance with connector application limits. Tests shall be conducted with all sheet steel and base steel thickness combinations for which the connection is intended. Test specimen strength and flexibility shall be determined in accordance with Sections 10.5 and 10.6 respectively.

8.3 Tension Tests

8.3.1 The following failure modes shall be identified when loading induces tension in the shanks of fasteners that connect steel as thin as 0.0149 in. (0.38 mm) to structural support members:

a. pull over
b. pull out
c. gross distortion.

8.3.2 The standard tension-test specimen for pull-over strength or for pull-out strength shall be specially formed from flat sheet stock used to produce the sheet steel product under consideration. See Figure 3(a) or 3(b). The specimen geometry shall serve as a generic model for profiled sheet steel. The connection support member thickness for the pull-over test shall be at least 1/8 in. (3 mm) in order to resist fastener pull-out with a minimal amount of symmetrical deformation from the test loads. Support members for standard pullover tests shall be in accordance with Table 3. For pull-out tests the field total sheet steel thickness connected to the support shall be permitted to be simulated by adding pieces of packing. See Figure 3(b).

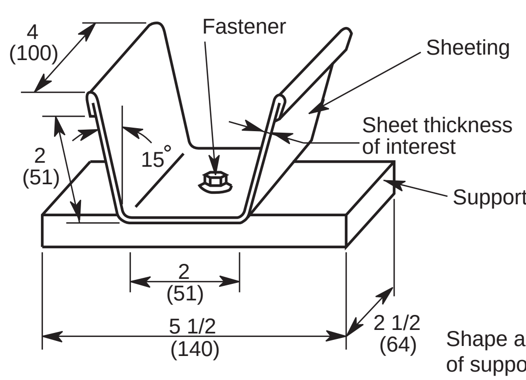
8.3.2.1 The standard tension-test fixture shall be designed for clamping of the test specimen and central loading along the axis of the fastener. See Figure 4.

8.3.2.2 The alternative tension-test fixture with a hook shall be permitted for tension pull-out specimens. Test fixture dimensions shall be in accordance with ASTM E1190. See Figure 5.

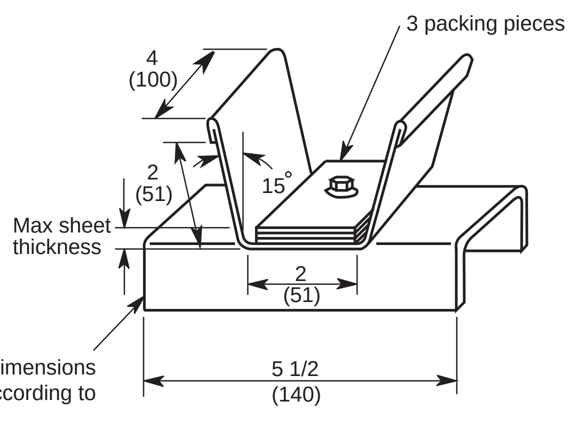
8.3.2.3 The influence of asymmetrical deformation of the sheet steel support members on pull-over strength shall be permitted to be tested with the generic standard test specimen by using the modified standard tension-test fixture. See Figure 6. Where the supporting member rotates, such as a C- or Z-shaped purlins or girts between lateral supports in metal buildings, the prying tension shall be considered.

Table 3
Standardized Support Members for Tension Pull-Over Tests

Thickness of Support Material, in. (mm)	t ≥ 1/4 (6.5)	t < 1/4 (6.5)	
Type of Support to be Used in Practice	All Types	Hot-Rolled Sections	Cold-Formed Sections, Hollow Sections, and Sheet Steel
Standardized Support to be Used in the Tests	Hot-Rolled Flat Steel: 2-3/8 x t (60 x t)	Hot-Rolled Angle: 1-5/8 x 1-5/8 x t (40 x 40 x t)	Cold-Formed Channel: 2-3/4 x 1-3/16 x t (70 x 30 x t)



(a) Standard Pull-Over Test Specimen
units – in. (mm)



(b) Standard Pull-Out Test Specimen
units – in. (mm)

Figure 3 – Standard Test Specimens

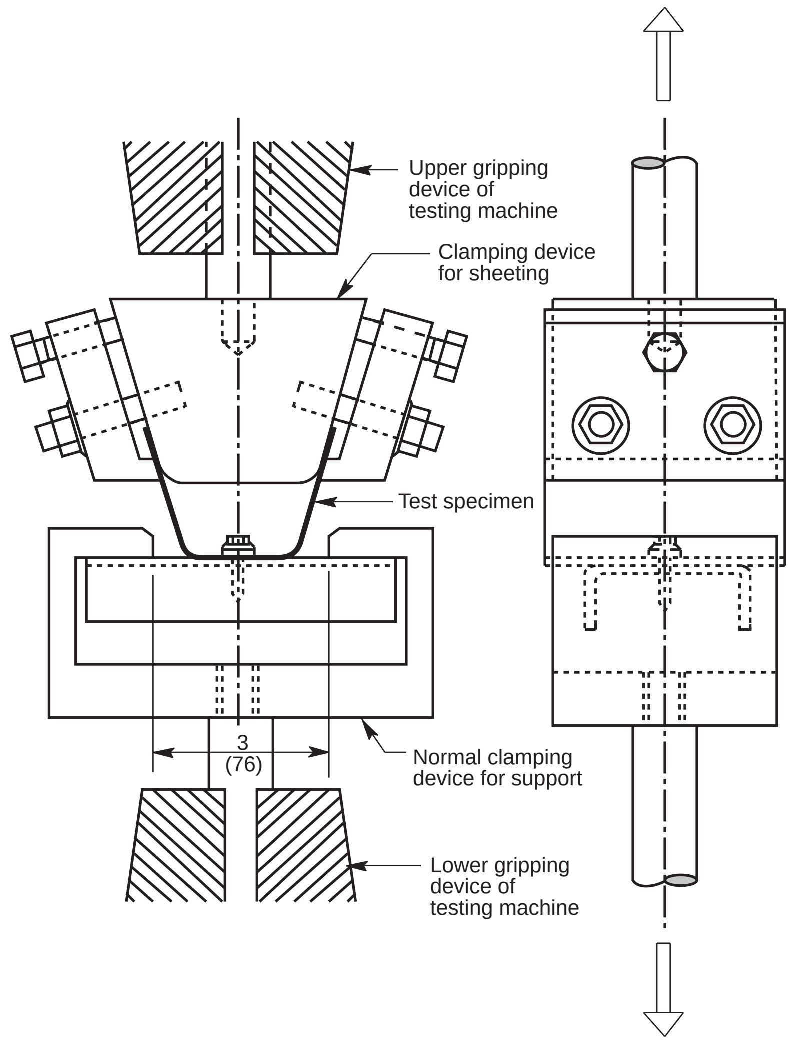


Figure 4 – Standard Tension Test Fixture

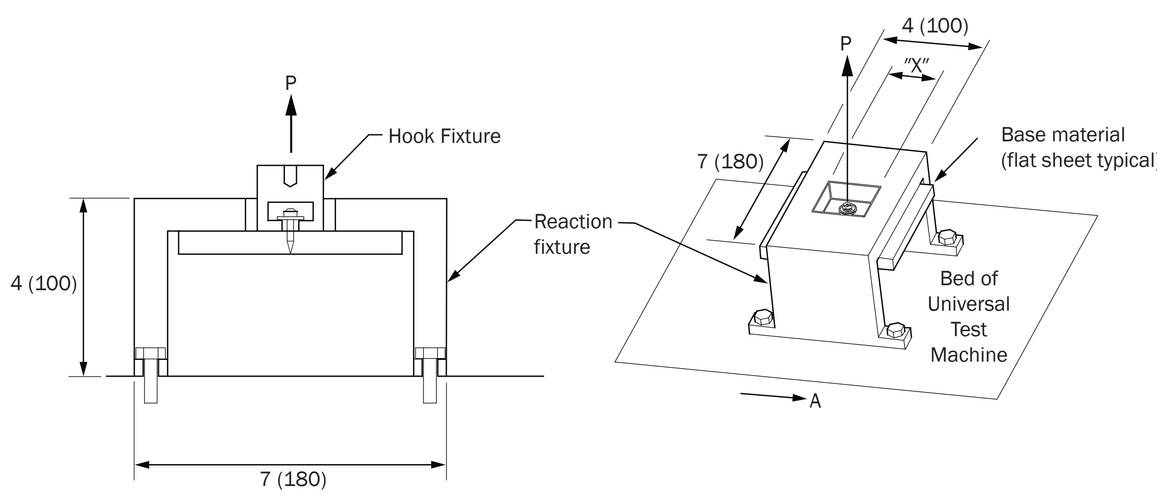


Figure 5 - Alternative Tension-Test Fixture with Hook, units – in. (mm)

User Note:
Reference ASTM E1190 Table 1 for test fixture dimension X as specified in Figure 5.

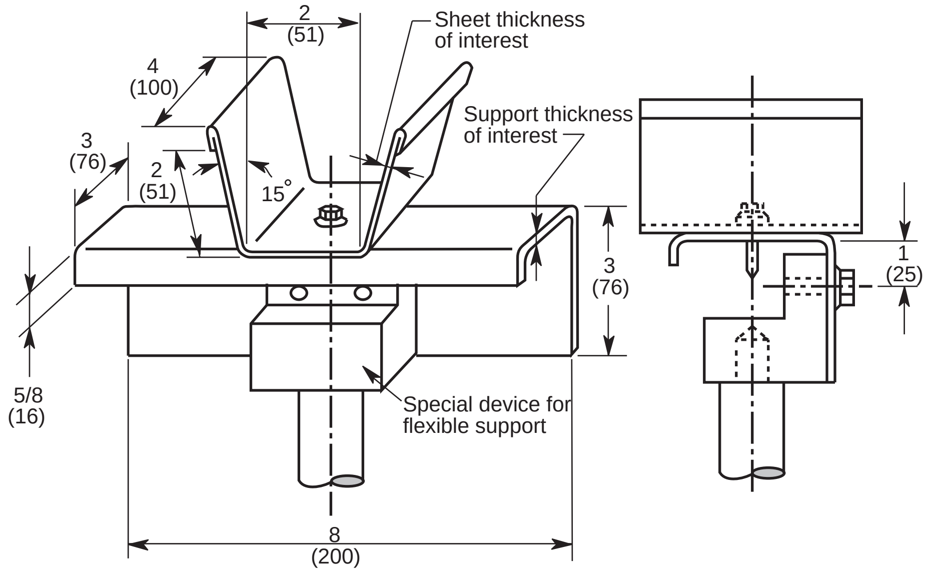


Figure 6 - Modified Standard Tension – Test Fixture for Influence of Flexible Support Members, units – in. (mm)

8.3.3 A specific standard tension-test specimen for pull-over strength of a profiled sheet , with width and length dimensions of 8 in. by 8 in. (200 mm by 200 mm), shall be cut from the sheet steel under consideration and drilled for four 1/2 in. (13 mm) diameter bolts located 6 in. (150 mm) apart. See Figure 7. The specimen shall be cut and drilled so that the location of the test fastener on the sheet profile corresponds to the location where the prototype has flexural tensile stresses in the region of the fastener, which will augment the tensile stresses caused by the fastener pull-over test.

8.3.3.1 The test fixture for a specific standard tension test shall consist of a stiff base plate assembly with four tapped holes located to match the holes in the test specimen. See Figure 7. The test specimen shall be clamped to the base with four 1/2 in. (12 mm) diameter bolts with 1-1/8 in. (29 mm) diameter by 3/32 in. (2.5 mm) thick washers under the bolt heads. Central loading shall be provided by a loading arm that is pin connected to the symmetric loading channel to which the sheet steel is fastened. The loading channel shall be permitted to be fabricated from the member used in the field connection or specially fabricated in accordance with the test objectives.

8.3.3.2 A modification to the test fixture shall be required to resist excessive deformation in thin flexible profiled sheet steel with relatively wide flat-widths. See Figure 8. Excessive deformation shall be prevented by stiffening angles attached to the base plate assembly. The angles shall be located a distance apart equal to the flat-width of test sheet steel.

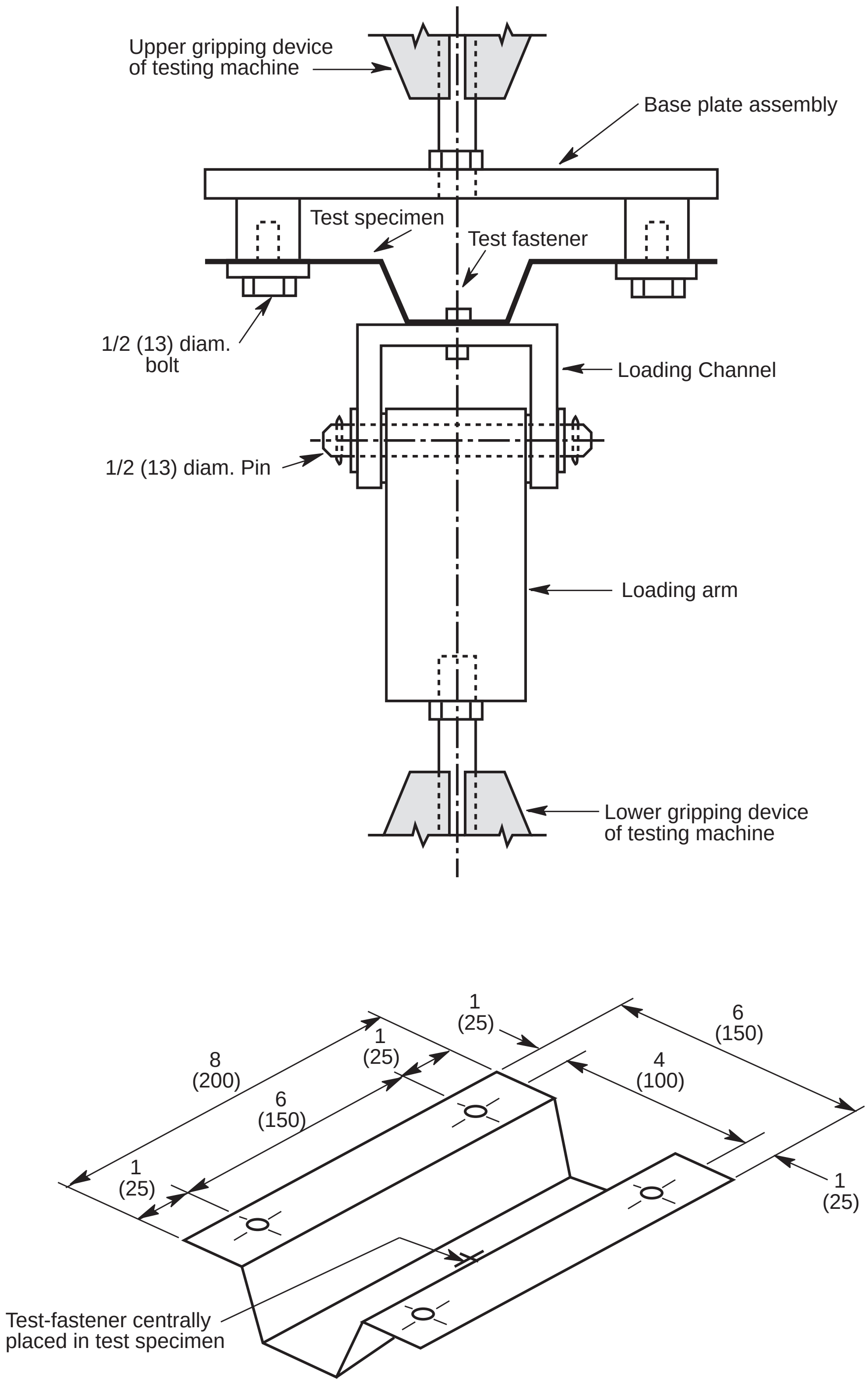
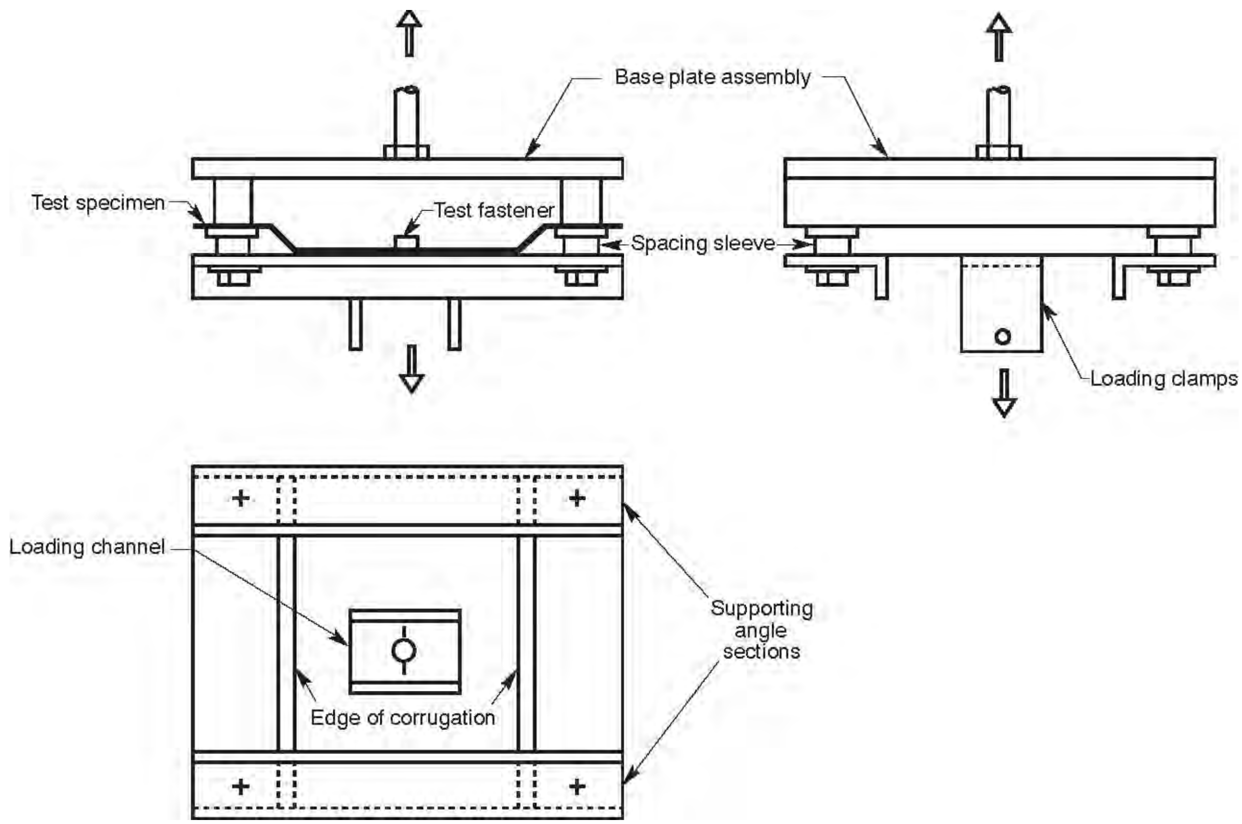


Figure 7 - Specific Standard Tension – Test Specimen and Fixture, units – in. (mm)



Bottom View

Figure 8 - Modified Specific Standard Tension – Test Specimen and Fixture With Location of Support Angles for Flexible Sheet Steel

8.3.4 The alternative tension-test specimen with a trapezoidal cross section shall be permitted to be utilized whenever detailed information about the sheet steel deformation is required, or where the prototype has flexural tension at the fastener. See Figures 9(a) and 9(b). The specimen shall consist of a segment of the test sheet steel with a centrally located test fastener that connects to a loading channel similar to that used for the standard tension test. The length of the specimen, L, shall be such that the flexural tension is at design value, and L is at least 12 times the flat-width of the corrugations, b.

The width of the specimen shall be 2 corrugations for trough fastening and 3 corrugations for crest fastening. Transverse stiffening straps approximately 3/4 in. (20 mm) wide by 1/16 in. (2 mm) thick shall be fastened across the specimen width to insure that the sheet profile is maintained during loading. The straps shall be located approximately $L_a/4$ from the mid length of the specimen.

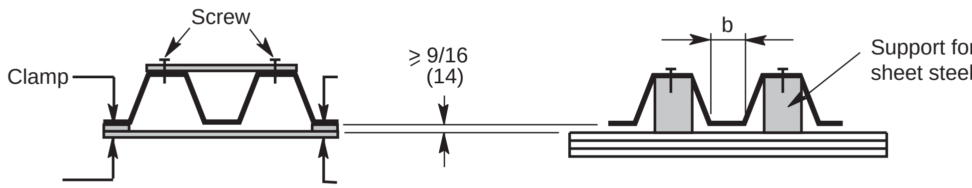
8.3.4.1 The test fixture shall provide for simple end supports of the test specimen. For tensile loading of a single fastener at midspan, a loading channel shall be used. See Figure 9(c). The test specimen and the test fixture shall provide the following proportions:

1) Span L_a equal to or greater than 6b, and less than $3M_u/P$, to prevent premature bending failure of the test sheet,

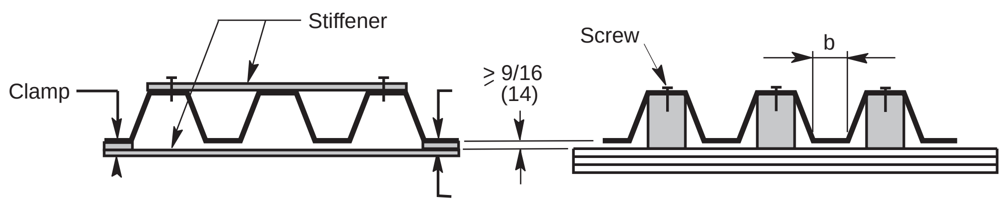
2) Specimen sheet steel support width, a, less than $L_a/6$, and

3) Transverse stiffening straps are located at a distance $L_a/4$ from the midspan.

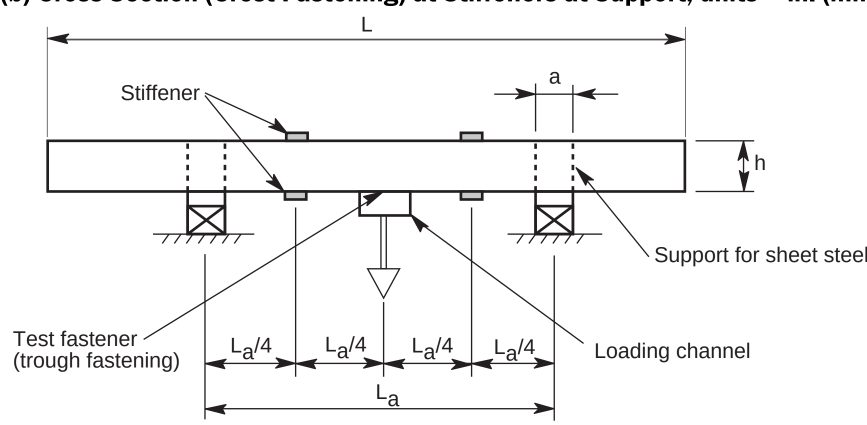
8.3.5 Where sheet steel is fastened to the support by a clip, as shown in Figure 9(d), or by a fastener near the edge of the underneath sheet so as to hide the fastener head, as shown in Figure 9(e), the test specimen shall be such that the panel lap is at the center of the width. All other specimen dimensions shall meet the requirements of Section 8.3.4. The test fixture requirements at stiffeners and sheet steel supports shall be the same as in Section 8.3.4.1.



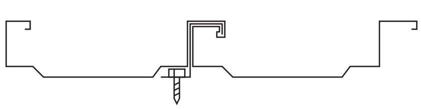
(a) Cross Section (Trough Fastening) at Stiffeners at Support, units – in. (mm)



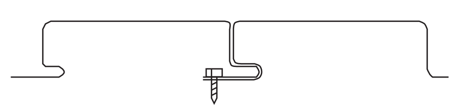
(b) Cross Section (Crest Fastening) at Stiffeners at Support, units – in. (mm)



(c) Test Fixture (Elevation)



(d) Clip Fastening



(e) Hidden Fastener

Figure 9 - Alternative Tension - Test Specimen and Fixture

8.3.6 A large-scale tension test capable of full-scale prototype testing of sheet steel connections shall be permitted to be conducted. See Figure 10. The test panel shall contain two beams (purlins or girts, or similar). The test panel shall be uniformly loaded over its surface by regulating the air pressure inside the chamber below the panel.

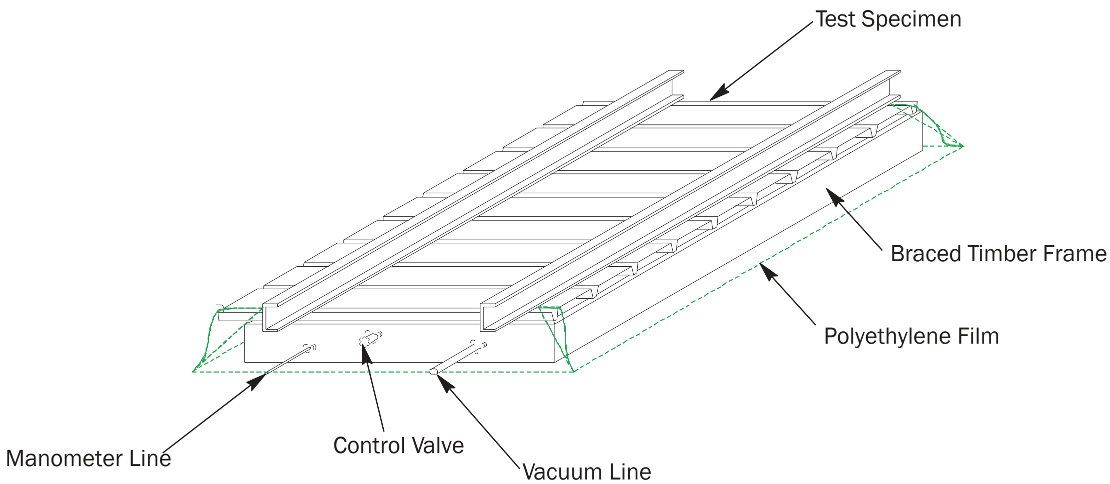


Figure 10 - Large Scale Tension Test

9. Test Procedures

9.1 General

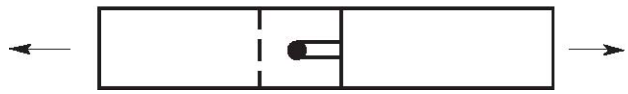
9.1.1 The speed of testing shall not be greater than that at which the relative displacement readings can be accurately taken. End grips of the testing machine shall be in alignment with the axis of the specimen test fixtures during loading.

9.1.2 When manual controlled tests are performed without computerized crosshead rate control and without a computer based data acquisition system, loading shall be applied in load increments of approximately one fifth of the estimated maximum load. When the maximum estimated load is approached, smaller increments shall be used. Each load increment shall be maintained for at least one minute (or until it has stabilized) before proceeding with the next increment. Loading shall continue until the load cannot be maintained, or until one or more fasteners have failed.

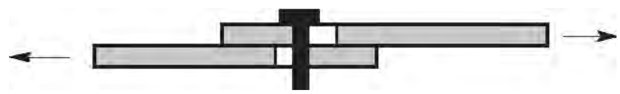
9.1.3 When a computerized test system is used with crosshead rate control capacity and data acquisition capability, the speed of testing as determined by the rate of separation of the testing-machine heads shall be 0.10 in. (approximately 3 mm) per minute or the rate caused by a loading rate of 500 pounds (2 kN) per minute.

9.2 Shear Tests

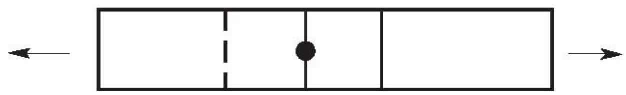
9.2.1 The speed of testing shall conform to Section 9.1.2 or 9.1.3 depending on the type of test system in use. For structural and stitch sidelap diaphragm connectors, once the ultimate lap shear capacity of the test specimen has been reached, the deformation or slip measurements corresponding to 40% of the ultimate lap shear capacity shall be measured on the load-displacement curve.



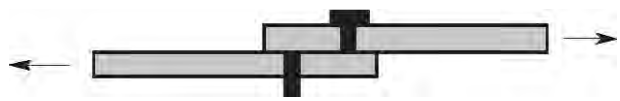
(a) Type I Failure
(End Shearing Failure)



(b) Type II Failure
(Bearing, Tearing, Piling Up)



(c) Type III Failure
(Tension Failure in Net Section)



(d) Type IV Failure
(Shearing of Fastener)

(e) Type V Failure
(Tilting, Pull-Over, and Pull-Out of Fastener)

Figure 11 - Lap-Joint Shear Test Failure Modes

9.2.2 The failure mode(s) shall be identified and recorded in accordance with the following classifications:

Type I - End failure, or longitudinal shearing of the sheet along two approximately parallel lines (Figure 11(a)).

Type II - Bearing tearing, or piling up of the thinner, or of both (equal thickness), sheet material in front of the fastener (Figure 11(b)).

Type III - Tension failure of one sheet in the net section (Figure 11(c))

Type IV - Shearing of the fastener (Figure 11(d))

Type V - Tilting and pull-out of fastener including sheet pull-over (Figure 11(e)).

Figure 11 shows only 1 fastener but it shall also be applicable for 2 or more fasteners.

9.3 Tension Tests

9.3.1 The speed of testing shall conform to Section 9.1.1 or 9.1.2 depending on the type of test system in use.

9.3.2 When deformation measurements are necessary, they shall be recorded at each loading increment and at the maximum load. If permanent-set measurements are necessary, a small preload (approximately 10 percent of the anticipated maximum load) shall be permitted to be used. After each loading increment, the load shall be reduced to the preload and the permanent set shall be recorded.

10. Calculations

Calculations to evaluate the test results and to determine the characteristic connection strength shall be made in accordance with Section F1 of AISI S100.

For diaphragm design, the strength and the stiffness/flexibility shall be calculated in accordance with 10.1 to 10.6.

10.1 Structural connector shear strength, Q_f, stitch sidelap connector shear strength, Q_s and structural connector pullover strength, P_{nov} shall be determined in accordance with F1.1 of AISI S100.

10.2 When tests include a range of variables, it shall be permitted to calibrate an equation for calculating the strengths of Q_f, Q_s and P_{nov}. The calibration shall be in accordance with F1.1 of AISI S100.

10.3 Safety factor (Ω) determined shall be less than or equal to and the resistance factor (ϕ) determined shall be greater than or equal to those presented in AISI S100 Table D5. Otherwise, full scale testing of the diaphragm system shall be required.

10.4 When tests include a range of variables, it shall be permitted to calibrate an equation for calculating the structural connector flexibility, S_f, and stitch sidelap connector flexibility, S_s. Calibration of the equation or the characteristic flexibility per fastener shall be permitted. The coefficient of variation (V_p) of the test results shall be less than those of the full scale diaphragm system stiffness tests used to confirm an analytical design.

10.5 Using standard shear test (Figure 1), test specimen strength shall be determined as:

$$Q_f \text{ (or } Q_s\text{)} = P_u/2 \tag{1}$$

where

Q_f, Q_s = Structural or stitch sidelap connector strength for sheet steel to base steel or sheet steel to sheet steel in diaphragms, respectively

P_u = Maximum connection strength

10.6 Using standard shear test (Figure 1), test specimen flexibility shall be determined as:

$$S_f = \frac{\Delta_g}{(N/2)(0.4Q_f)} \tag{2a}$$

$$S_s = \frac{\Delta_g}{(N/2)(0.4Q_s)} \tag{2b}$$

where

S_f = Structural connector flexibility for sheet to base steel in diaphragm

S_s = Stitch connector flexibility for sheet to sheet steel in diaphragm
Δ_g = Total elongation of extensometer gage length ℓ_g at 40 percent of P_u
N = Number of fasteners in lap-joint connection
= 2 for setup 1 as illustrated in Figure 1 (a)
= 4 for setup 2 as illustrated in Figure 1 (b)
Q_f = Structural connector strength for sheet to base steel in diaphragm
Q_s = Stitch connector strength for sheet to sheet steel in diaphragm

11. Test Report

11.1 The objectives of the test series shall be stated at the outset of the report so that the necessary test results, such as the maximum load per fastener, the flexibility of the connection, and the failure mode, are identified.

11.2 The type of tests, the testing organization, and the dates on which the tests were conducted shall be included in the documentation.

11.3 The test unit shall be fully documented, including:

1) the measured dimensions of each specimen,
2) identification data for the fasteners and accessories such as washers. Fastener data shall include the name of the manufacturer, designation or type, dimensions, number of threads, including unthreaded length or imperfect threads below head, and the major and minor diameters in the threaded region.
3) the details of fastener application including pre-drilling, tightening torque, and any unique tools used in the operation, and
4) the results of the sheet-type tension tests including yield stress, tensile strength, and elongation to failure. The location and orientation of the sheet-tension coupons shall also be given.

For pull-out tests additional data shall indicate the drill-point diameter and length of flutes if self-drilling screws are used. Otherwise, the diameter of the pilot drill used shall be stated. Washers or washer-head data shall include diameter, thickness, configuration, and material. Sealing washers shall additionally define the type, material, and dimensions of sealant.

11.4 The test set-up shall be fully described including the testing machine, the specimen end grips or supports and the devices used to measure deformation.

11.5 The test procedure shall be fully documented including the rate of loading and the load increments.

11.6 In accordance with the test objectives stated by the responsible engineer, the report shall include a complete documentation of all applicable test results for each specimen such as the load-deformation curve, the maximum load, and the mode of failure. The report shall also include the necessary calculations for the characteristic connection strength per fastener and the connection flexibility for the test unit. Calculations for reduction of the test strength (corresponding to the specified minimum yield stress of the steel sheet product) shall also be included when applicable.

Commentary on S905-08

Test Methods for Mechanically Fastened Cold-Formed Steel Connections

The continued introduction of new and different mechanical fasteners increases the need for standardized tests for cold-formed steel connections. Standard test specimens, fixtures, and procedures facilitate the exchange of information vital to understanding the behavior of a variety of fasteners with diverse properties by providing a basis for comparing strength and deformation measurements. This test standard provides shear and tension performance test procedures for determining the strength and deformation of mechanically fastened connections of cold-formed steel members including diaphragm applications. Connections involving gypsum wallboard, wood and Oriented Strand Board (OSB) are outside the scope of this test method. These test methods provide the requirements for evaluating mechanically fastened connections for cold-formed steel members in buildings designed according to AISI S100 (2007) and related building codes. These performance test procedures are based extensively on test methods used successfully in the past (References 2-4).

The failure modes that can be identified by this test procedure include tension pull-out failure, and shear failures: end shearing (Figure 11(a)), bearing, tearing, piling up (Figure 11(b)), tension failure on net section (Figure 11(c)), shearing of fastener (Figure 11(d)), and tilting, pull-over, and pull-out of fasteners (Figure 11(e)). It is critical to properly identify the failure modes during the tests. Practically, certain failure modes may prohibit the use of a connection regardless of its strength.

Standard and alternative test setup of specimens and fixtures are provided in Standard Section 8 for lap-joint shear and tension tests. Test procedures and data recording are generally not affected by the modified specimens and fixtures used for alternate tests, but these alternate tests represent the state-of-the-art with respect to mechanical fastener testing.

The standard tension-test fixture described in Standard Section 8.3.2.1 is designed for the convenience of clamping the test specimen and centering the load along the axis of the fastener.

The dimensions of the asymmetrical support member as shown in Figure 6 of the Standard simulate mechanical fastenings to cold-formed C-Sections, Z-Sections and angles. References 5 and 10 provide information on how prying tension is considered.

For the full-scale prototype test as shown in Figure 10 of the Standard, failure can occur in the panel connections to the beams. Only the nominal tension forces, which do not include the prying forces caused by the rotations of the C- and Z-shaped purlins or girts, can be computed from the known value of the load acting on the panel surface at failure. A description of prying forces is given in Reference 10. The arrangement illustrated in Figure 9 is fully described in References 3, 5, 6, and 7. The beam span and the purlin spacing should match those of the prototype of Reference 10 within ±20 percent in order to properly simulate the effects of prying action.

Calculations to evaluate the test results and to determine the characteristic connection strength should be made in accordance with F1 of AISI S100 (2007). The test result evaluation described in the Standard is based on procedures provided in References 8 and 9. Standard Equations (2a) and (2b) for determining the test specimen flexibility are developed based on Reference 11.

The determination of the individual fastener lap-joint shear strength is independent of which test setup is used. The test setups with two and four fasteners are essentially the same. The two fastener setup shown in Figure 1 (a) has one base steel component and one sheet steel component. The four fastener setup shown in Figure 1 (b) uses two base steel components with one sheet steel component bridging across both base steel components. The different test setups are the same from a load distribution standpoint, with the four fastener test specimen offering fabrication and test specimen efficiency. The individual fastener lap-joint shear strength is the total connection strength divided by two in either case.

The determination of the individual fastener lap-joint shear flexibility however, is dependent on which test setup is used. The equation denominator term (N/2) takes this difference into account.

References

1. American Iron and Steel Institute (2007), AISI S100-07, *North American Specification for the Design of Cold-Formed Steel Structural Members*, Washington, DC, October 2007.
2. ECCS Committee 17 Report 21 "European Recommendations for the Testing of Connections in Profiled Sheeting and Other Light Gauge Steel Components", May 1978
3. Fraczek, J. "Mechanical Connections in Cold-Formed Steel; Comprehensive Test Procedures and Evaluation Methods," Research Report No. 359, Dept. of Structural Engineering, Cornell University, Ithaca, NY., May 1976
4. Fraczek, J. "Development of Comprehensive Test Procedures for Connections in Cold-Formed Steel and Appropriate Evaluation Methods.", Draft Report, Dept. of Structural Engineering, Cornell University, Ithaca, NY., Oct 1974
5. LaBoube, R.A., Golovin, M., Montague, D.J., Perry, D.C., and Wilson, L.L., "Behavior of Continuous Span Purlin Systems," *Ninth International Specialty Conference on Cold-Formed Steel Structures,* St. Louis, MO., November, 1988, University of Missouri-Rolla.
6. Pekoz, T., and Soroushian, D., "Behavior of C- and Z Purlins Under Wind Uplift, "*Sixth International Specialty Conference on Cold-Formed Steel Structures,* St. Louis, MO., November, 1982, University of Missouri-Rolla
7. LaBoube, R.A., "Laterally Unsupported Purlins Subjected to Uplift," Metal Building Manufacturers Association, December, 1983
8. Pekoz, T., and Hall, B., "Probabilistic Evaluation of Test Results, *"Ninth International Specialty Conference on Cold-Formed Steel Structures,* St. Louis, MO., November, 1988, University of Missouri-Rolla.
9. *Cold-Formed Steel Design Manual,* American Iron and of Steel Institute, 2002 Edition
10. Haussler, R.W., "Theory of Cold-Formed Steel Purlin/Girt Flexure," *Ninth International Specialty Conference On Cold-Formed Steel Structures,* St. Louis, Missouri, November, 1988, University of Missouri – Rolla
11. Steel Deck Institute (SDI) Diaphragm Design Manual, 3rd Edition, September 2004 and Appendix VI, November 2006.

AISI S906-08

Standard Procedures for Panel and Anchor Structural Tests

1. Scope

This procedure extends and provides methodology for interpretation of results of tests performed according to ASTM E1592.

2. Referenced Documents

The following documents or portions thereof are referenced within this Standard and shall be considered as part of the requirements of this document.

a. American Iron and Steel Institute (AISI), Washington, DC:

S100-07, *North American Specification for the Design of Cold-Formed Steel Structural Members*

b. ASTM International (ASTM), West Conshohocken, PA:

A370-07b, Standard Test Methods and Definitions for Mechanical Testing of Steel Products

E6-07b, Standard Terminology Relating to Methods of Mechanical Testing

E1592-01, Standard Test Method for Structural Performance of Sheet Metal Roof and Siding Systems by Uniform Static Air Pressure Difference

IEEE/ASTM-SI-10-02, American National Standard for Use of the International System of Units (SI): The Modern Metric System

c. U. S. Army Corps of Engineers:

CEGS-07416, Guide Specification for Military Construction, Structural Standing Seam Metal Roof (SSSMR) System, 1995

d. Factory Mutual, Corporate Offices, 1301 Atwood Avenue, P.O. Box 7500, Johnson, RI 02919:

FM4471, Approval Standard for Class 1 Metal Roofs, 1995

3. Terminology

Where the following terms appear in this Standard they shall have the meaning as defined herein. Terms not defined in Section 3 of this Standard, AISI S100 or ASTM E6 shall have the ordinary accepted meaning for the context for which they are intended.

3.1 Refer to Section 3, ASTM E1592.

3.2 Additional or Modified Terminology

Clip. A single or multiple element device that frequently attaches to one edge of a panel and is fastened to the secondary structural members with one or more screws.

Field. Area that is not included in high pressure edge strip conditions. For purposes of the test, a field condition is modeled when the pan distortions are independent of end and edge restraint.

Pan. Relatively flat portion of a panel between ribs.

Tributary area. Area directly supported by the structural member between adjacent supports.

Trim. Sheet metal used in the finish of a building especially around openings, and at the intersection of surfaces such as roof and walls.

Maximum load. Difference in static air pressure at which failure of the specimen occurs, expressed in load per unit area, and is further defined as the point where the panel system cannot sustain additional loading.

*Unlatching failure. D*isengagement of a panel seam or anchor that occurs in an unloaded assembly due to permanent set or distortion that occurred when the assembly was loaded. This permanent set is not always detectable from readings taken normal to the panel. It is deemed to be a serviceability limit state until a strength limit state occurs, as defined in maximum load.

4. Summary of the Test Method

4.1 Refer to the requirements of Section 4, ASTM E1592.

5. Significance and End Use

5.1 Refer to the requirements of Section 5, ASTM E1592.

5.2 The end use of the procedure is the determination of allowable load carrying capacity of panels and/or their anchors under gravity or suction loading for use in a design procedure.

6. Test Apparatus

6.1 Refer to the requirements of Section 6, ASTM E1592.

7. Safety Precautions

7.1 Refer to the requirements of Section 7, ASTM E1592.

8. Test Specimens

8.1 Refer to the requirements of Section 8, ASTM E1592.

8.2 Edge seals shall not contain attachments that restrict deflection of the test panel in the field in any way. No additional structural attachments that would resist deflection of the field of the test panels shall be permitted.

8.2.1 The test panel ribs shall be installed parallel to the long side of the test chamber.

8.3 Number of Tests

8.3.1 Tests shall use minimum thickness of support members (secondary structures) and maximum panel span. If results are to be interpolated for other values, the other extremes shall be tested in order to justify an interpolation procedure.

8.3.2 Tests shall be conducted to evaluate the field condition.

8.3.3 The minimum number of spans shall be as defined in Table 1.

9. Calibration

9.1 Refer to the requirements of Section 9, ASTM E1592.

10. Procedures

10.1 Refer to the requirements of Section 10, ASTM E1592.

11. Test Evaluation

11.1 Safety factors and resistance factors shall be determined in accordance with Section D6.2.1 of AISI S100.

11.2 If a separate test series is performed to evaluate edge conditions and the results exceed the field case by greater than one standard deviation, a separate design allowable shall be permitted to be established for edge conditions.

11.3 A qualified design professional shall analyze deflections and permanent set data to assure that deflections and permanent set are acceptable at service loads.

TABLE 1
Minimum Number of Equal Spans to Comply with 8.3*

Span Length, L ft. (m)	Ends with Crosswise Restraint**		
	2 end restraints	1 end restraint	No end restraints
12 ft-0 in. (3.7m) or more	2	2	2
Below 12 ft-0 in. (3.7 m) to 8 ft-0 in. (2.4 m)	3	2	2
Below 8 ft-0 in. (2.4 m) to 6 ft-0 in. (1.8 m)	4	3	2
Below 6 ft-in. (1.8 m) to 5 ft-0 in. (1.5 m)	5	3	2
Below 5 ft-0 in. (1.5 m) to 4 ft-0 in. (1.2 m)	24/L (7.3/L)	3	3
Below 4 ft-0 in. (1.2 m) to 3 ft-4 in. (1.0 m)	24/L (7.3/L)	4	3
Below 3 ft-4 in. (1.0 m) to 3 ft-0 in. (0.9 m)	24/L (7.3/L)	4	4
Below 3 ft-0 in. (0.9 m) to 2 ft-6 in. (0.8 m)	24/L (7.3/L)	5	4
Below 2 ft-6 in. (0.8 m) to 2 ft-0 in. (0.6 m)	24/L (7.3/L)	5	5
Below 2 ft-0 in. (0.6 m)	24/L (7.3/L)	1+8/L (1+2.4/L)	10/L (3.0/L)

* Count fractional spans as whole numbers, that is, for L=4 ft-9 in. (1.4 m), 24/4.75 =5.05(or 7.3/1.4=5.2), use 6 spans.

** L is measured in feet (when L is measured in meters, the corresponding formula in the parentheses is used).

12. Test Report

12.1 Refer to the requirements of Section 11, ASTM E1592.

12.2 Report the nominal strength of the standing seam roof panel system as follows:

12.2.1 For E1592, the nominal strength shall be the ultimate load as defined by that test procedure.

12.2.2 For FM 4471, the nominal strength shall be the minimum uplift pressure recorded for windstorm classification achieved.

12.2.3 For CEGS-07416, the nominal strength shall be the ultimate load.

12.3 If intermediate values are to be calculated for different spacings of anchors or secondary structures, the basis of the interpolation shall be stated in the report. If the failure modes are different on any two tests, interpolation between these two tests shall be permitted provided the lower bounds of the two failure modes are used.

12.4 The design professional shall include in the report the observation as to the acceptability of deflections and permanent set data at service loads.

Commentary on AISI S906-08

Standard Procedures for Panel and Anchor Structural Tests

1. Scope

The scope of the Procedure is for testing single skin panel systems. The procedure is based on ASTM E1592-01 with specific additions to define the required safety factors for a design procedure. Edge strip detail confirmation is permitted by the test method.

2. Reference Documents

The standards, ASTM E1592-01, U. S. Army Corps of Engineers CEGS-07416(1995), and Factory Mutual 4471 (1995) have been used in the development of this procedure.

3. Terminology

To promote accuracy and understanding, frequently used terms need mutual understanding. This list includes the terms from ASTM E1592-01 with additions and modifications.

5. Significance and End Use

Currently, there are several organizations that have test procedures to determine product performance, but the procedures are limited to one product configuration and do not have provisions to provide the basis for a complete design procedure covering the evaluation of a safety factor for a range of product configurations. Therefore, this Standard Procedure was developed.

6. Test Apparatus

The apparatus defined in this section is specific enough to accomplish the purpose, yet broad enough to allow many facilities to perform tests. The size of the specimen is the most important criteria. Whether or not the apparatus consists of two sections with the specimen in between is not a major issue.

Measurement of rib spread has dubious value except when seam disengagement is the failure mechanism. In that case, measurements tend to substantiate the failure mechanism.

7. Safety Precautions

In addition to other precautions, care must be exercised in taking the deflection readings required in this procedure.

8. Test Specimens

The size of a test specimen has been found to be an important element in demonstrating product performance. Minimum sizes are defined, but larger sizes are allowed. It is understood that many products are offered to the market that have insufficient usage to justify a large test program yet proof of performance to some degree is required. The procedure is developed to allow a single test with a corresponding penalty due to the reduced degree of demonstrated reliability with only a single test. The procedures of Chapter F of AISI S100 provide for the reward/penalty relationship developed with increasing number of tests and the associated coefficient of variation.

Minimum specimen size is as required in ASTM E1592-01 with the addition of the specimen configuration with both ends open. This is consistent with the 1995 edition of E1592 and continues to approve tests previously conducted according to that protocol. Reports from manufacturers using these test results suggest that these tests formed the basis of adequate designs. The minimum specimen length of 24 ft. (7.3 m) for the condition of constraint at both ends is consistent with the requirements of Factory Mutual (FM) Procedure 4471 (1995). However, in the FM tests, panels may be fastened down at all edges and it is termed a field test. It is the manufacturer's option to fasten the ends down or leave them open. The details of the FM test. may not meet the ASTM E1592-01 Number of Equal Spans requirement in some cases. A purlin space of 5 ft. (1.5 m) requires 5 spans with both ends restrained. If one end is left free, the FM test will meet ASTM E1592-01. The application is also different in many cases because FM tests may be run with both ends restrained and this is used as a field test. Different results may be obtained when using the two variations of panel end restraints in the test procedure that are allowed by ASTM E1592-01.

When totaling the number (n) of anchors tested for evaluation of C_p under Section D6.2.1 of AISI S100, it is permissible to include all fasteners with the same tributary area as that associated with a failed anchor instead of merely totaling the number of physical tests run on a complete assembly. When totaling the number (n) of panels tested for evaluation of C_p under Section D6.2.1 of AISI S100, it is permissible to include all panels with the same tributary area as that associated with a failed panel instead of merely totaling the number of physical tests run on a complete assembly

Consideration is given to the minimum spacings and material thicknesses. If allowables developed under this procedure are intended to be used in a design procedure that encompasses different secondary structural support spacings or thinner sections for anchors to attach to, the extremes must be tested in order for interpolation to be valid. This precedent is established in the AISI S908-08, Base Test Method (2008) for validating the performance of purlins braced by standing seam roof panels.

10. Procedures

The procedures for loading the specimen, while not complicated, need to be defined consistent with other existing and recognized standards. A significant difference between this procedure and the AISI S908 (2008) is the return to zero load after each load increment.

11. Test Evaluation

See Section D6.2.1 of the *Commentary* for AISI S100 (2007).

12. Test Report

The definition of items to be included in the report includes the typical list of failure loads and plots of load versus deformation. Of paramount importance is the calculation of the resistance factor and safety factor of design strength or allowable design strength for panels and anchors. This procedure is an addition to those required in ASTM E1592-01. If interpolation is to be a part of the resulting design process, then appropriate interpolation procedure should be set forth in the report.

References

American Iron and Steel Institute, S100-07, *North American Specification for the Design of Cold-Formed Steel Structural Members*, 2007.

American Iron and Steel Institute, S100-07-C, *Commentary on North American Specification for the Design of Cold-Formed Steel Structural Members*, 2007.

American Iron and Steel Institute, S908-08, Base Test Method for Purlin Supporting a Standing Seam Roof System, AISI Standards for Test Procedures, 2008

ASTM International, E1592-01, Standard Test Method for Structural Performance of Sheet Metal Roof and Siding Systems by Uniform Static Air Pressure Difference, 2001.

U. S. Army Corps of Engineers, CEGS-07416 (1995), Guide Specification for Military Construction, Structural Standing Seam Metal Roof (SSSMR) System, July 1995

Factory Mutual Research (1995), "Approval Standard for Class I Panel Roofs, Class Number 4471", August 1995

AISI S907-08

Test Standard for Cantilever Test Method for Cold-Formed Steel Diaphragms

1. Scope

This Standard applies to framed roof or floor cold-formed steel deck diaphragm construction.

1.1 This test method covers the determination of the nominal diaphragm web shear strength and web shear stiffness, and flexibility, for framed roof or floor cold-formed steel deck diaphragm construction.

1.2 This Standard consists of Sections 1 through 14 inclusive.

2. Referenced Documents

The following documents or portions thereof are referenced within this Standard and shall be considered as part of the requirements of this document.

a. American Iron and Steel Institute (AISI), Washington, DC:

AISI S100-07, *North American Specification for the Design of Cold-Formed Steel Structural Members*

AISI S904-02, Standard Test Methods for Determining the Tensile and Shear Strength of Screws, Reissued in 2007

AISI S905-08, Test Method for Mechanically Fastened Cold-Formed Steel Connections

b. ASTM International (ASTM), West Conshohocken, PA:

ASTM A370-07a, Standard Test Methods and Definitions for Mechanical Testing of Steel Products

ASTM C33-03, Standard Specifications for Concrete Aggregates

ASTM C39/C39M-05e1, Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens

ASTM C330-05, Standard Specification for Lightweight Aggregates for Structural Concrete

ASTM C332-99, Standard Specification for Lightweight Aggregates for Insulating Concrete

ASTM C495-99a, Standard Test Method for Compressive Strength of Lightweight Insulation Concrete

ASTM E6-07b, Standard Terminology Relating to Methods of Mechanical Testing

IEEE/ASTM SI-10-02, American National Standard for Use of the International System of Units (SI): The Modern Metric System

c. American Welding Society (AWS), Miami, FL:

AWS D1.1/D1.1M-2006, Structural Welding Code - Steel

AWS D1.3-1998, Structural Welding Code - Sheet Steel

3. Terminology

Where the following terms appear in this Standard they shall have the meaning as defined herein. Terms not defined in Section 3 of this Standard, AISI S100, A370 or ASTM E6 shall have the ordinary accepted meaning for the context for which they are intended.

Base Steel Thickness. The thickness of bare steel exclusive of all coatings.

Bare Frame. Steel members assembled to form the test frame of the diaphragm without the diaphragm web of steel deck panels installed onto the frame.

Cellular Deck Panels. Fluted steel deck panels, resistance welded to an essentially flat sheet or another fluted steel deck panel at a manufacturing facility.

Composite Slab. A steel deck system with a structural concrete fill placed onto the steel deck panels. The steel deck panels typically have web embossments or other shear connection devices to develop mechanical bond between the steel deck and the structural concrete fill so that the concrete and steel deck panels compositely resist applied vertical loads.

Configuration. Each configuration consists of one deck profile cross-section, one type of deck steel, one type and spacing of connections to vertical load supports, and one type of side seam fasteners between panels. Within the configuration, the thickness of the deck, spacing of side seam fasteners, and deck span can be varied.

Diaphragm. Roof, floor or other horizontal or near horizontal membranes or bracing systems that transfer in-plane forces to the lateral force resisting system, analogous to a horizontal girder with interconnecting deck panels acting as the girder web. Intermediate joists or beams act as web stiffeners and provide vertical load support. Perimeter steel beams or perimeter concrete or masonry elements with reinforcement act as girder flanges. Diaphragms under this Standard are bare steel decks with or without superimposed composite slabs or non-composite slabs.

Fluted Deck Panels. Steel deck panels with flanges in various patterns alternating from top to bottom.

Full Frame. Steel deck panels installed and attached to a bare frame to form the diaphragm test specimen.

Non-Composite Slab. A steel deck system with a concrete fill placed onto the steel deck panels, with the vertical loads on the assembly designed to be carried without composite behavior between the steel deck panels and concrete.

Plain Steel Deck. A steel deck system consisting of steel deck panels without concrete fill.

Steel Deck Panels. Sheets of steel, cold-formed into fluted or cellular decks with specified width and variable length.

4. Symbols

F = Shear flexibility of the diaphragm web as determined from test measurements

G′ = Shear stiffness of the diaphragm web as determined from test measurements

L_v = Span of steel deck taken as the distance between lines of fasteners connecting steel deck to supports

P = Applied load to test frame
P_d = Load P at which the diaphragm stiffness is determined
P_{fd} = Load P from testing of the bare test frame at the deflection equal to the deflection for load of $0.4P_{max}$ for stiffness
P_{fn} = Load P from testing of the bare test frame at the deflection equal to the deflection for load P_{max} for strength
P_{max} = Maximum applied load P to test frame
P_n = Load P for determining nominal unit shear strength
S_n = Nominal unit web shear strength of the diaphragm
a = Length of diaphragm test frame (See Figure 1)
b = Depth of diaphragm test frame and dimension parallel with load P (See Figure 1)
t = Base steel thickness of steel deck elements of diaphragm web
t_c = Thickness of fill material above the top of the steel deck
Δ_d = Net shear deflection of diaphragm at load $0.4P_{max}$
Δ_n = Net shear deflection of diaphragm at any load level
$\Delta_{1,2}$ = Displacement measured at points 1 or 2 (See Figure 2)
$\Delta_{1,2,3,4}$ = Displacement measured at points 1, 2, 3 or 4 (See Figure 3)

5. Units of Symbols and Terms

Any compatible system of measurement units shall be permitted to be used in this Standard, except where explicitly stated otherwise. The unit systems considered shall include U.S. customary units (force in kips and length in inches) and SI units in IEEE/ASTM SI-10 (force in Newtons and length in millimeters).

6. Precision

6.1 Loads shall be recorded within 2 percent of the expected diaphragm strength during the application of test loads.

6.2 Displacements shall be recorded to a precision of 0.01 in. (0.25 mm).

7. Full Frame Test Assembly

The full frame test assembly shall consist of a rectangular steel frame upon which the steel deck is placed and fastened using fasteners spaced as intended for end-use. The depth of the test frame shall be a multiple of the width of the steel deck panel, and the length of the test frame shall be a multiple of the steel deck intended end-use span. The ratio of diaphragm length to depth or depth to length shall not be less than 75% nor greater than 133%. The plan dimensions of a full frame test assembly shall be such that five or more steel deck panels are required to cover the test frame depth. The test assembly shall not be less than 12 ft. (3.6 m) in depth or length. The test diaphragm shall be representative of the end-use construction including the profile of the steel deck, deck thickness and tensile properties, span, L_v, fastener type, size and pattern, and support thickness and type. If edge transfer angles or profiled end closure elements are used for shear transfer of the deck to the frame members, they shall be included in the test assembly.

If required, interior purlins shall be used to carry the steel deck. One edge framing member shall serve as a horizontal load beam (See beam AB in Figure 1). The corners opposite to the ends of the load beam shall serve as the horizontal support to resist the loads applied to the load beam. The vertical support of the load beam shall be on rollers so that it can move freely horizontally (See Figure 1). Deck spans shall also be permitted to be parallel to the load beam. Out-of-plane movement of the load beam shall be minimized by any means that does not restrict the in-plane movement of the test assembly.

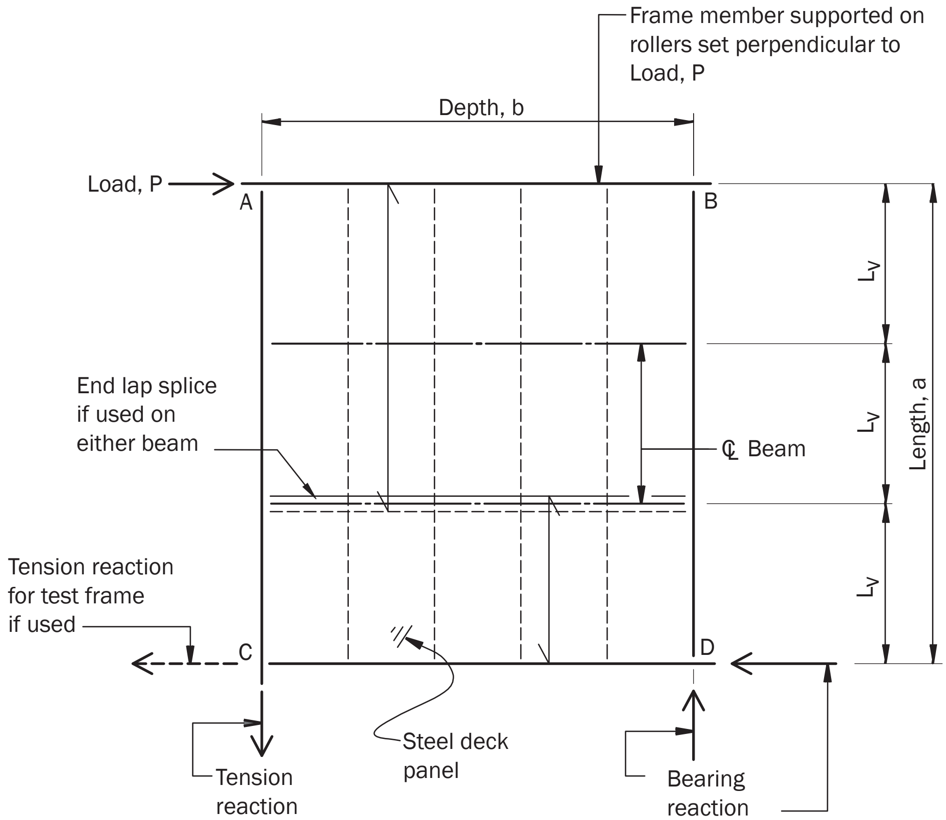
User Note:
To minimize the out-of-plane movement, apply the load to the load beam as close as practical to the shear center of the test assembly, apply an out-of-plane load to the load beam to offset the net vertical uplift, or apply a vertical restraining device (such as a roller to the frame beam).

The means used to restrain the vertical movement shall be in place during the bare frame test specified in Section 8.

Perimeter frame member ends shall be interconnected using angles or other devices to transfer developed axial forces into frame supports. Purlins shall be connected with bolted clip angles to perimeter frame members or other means to minimize bending moment transfer at member ends.

It shall be permitted to fasten the steel deck panels to the framing members parallel to the flutes at a spacing to be sure that the failure mechanism of the diaphragm occurs in the field of the test specimen web.

The end lap splice as shown in Figure 1 shall be permitted to be optional, but the test engineer shall require end laps if end lap performance is critical to the test evaluation.



Note: Reaction to axial load in member CD is permitted to be a tension reaction at point C.

Figure 1 - Test Frame Layout

User Note:
As indicated in Figure 1, it is acceptable, but not mandatory to place an end lap splice on at least one interior bearing to evaluate the impact of a splice on the strength and stiffness of the diaphragm type.

8. Testing a Bare Frame Assembly

The test frame without steel deck panels attached shall be subjected to the diaphragm load test procedure to determine the strength and stiffness of the bare frame as described for the tests of the full test assembly. The load beam member shall support at least the dead load that will be on the member when testing the full frame assembly.

9. Testing a Full Frame Test Assembly

The loading sequence to the maximum applied load, P_{max}, shall provide at least ten sets of load and displacement readings prior to reaching P_{max}. An initial load of 0.05 times the anticipated P_{max} shall be applied. The sets of readings shall be with approximately equal load increments. The rate of loading shall be such that P_{max} is achieved in not less than 10 minutes. Loads shall be applied with a calibrated load system. The weight of the specimen and load apparatus shall be accounted for, if it is anticipated that the weights will affect the results. Displacements shall be measured with dial gages or other devices. At load levels of approximately one quarter and one half of the estimated maximum load, the load shall be lowered to the initial load and the recovery of the diaphragm shall be recorded after 5 minutes.

10. Analysis of Diaphragm Test

10.1 Determination of Nominal Diaphragm Web Shear Strength

The nominal diaphragm web shear strength, S_n, which is the shear load per unit length across the full frame test, shall be calculated in accordance with Eq. 1:

$$S_n = \frac{P_n}{b} \tag{1}$$

where

$$P_n = P_{max} - P_{max}\left(\frac{P_{fn}}{P_{max}} - 0.02\right), \qquad \text{if } \frac{P_{fn}}{P_{max}} > 0.02 \tag{1a}$$

$$P_n = P_{max}, \qquad \text{if } \frac{P_{fn}}{P_{max}} \le 0.02 \tag{1b}$$

where

P_{max} = Maximum applied load P to test frame

P_{fn} = Load P from testing of the bare frame at the deflection equal to the deflection for load of P_{max} for strength

b = Depth of diaphragm test frame and dimension parallel with load, P

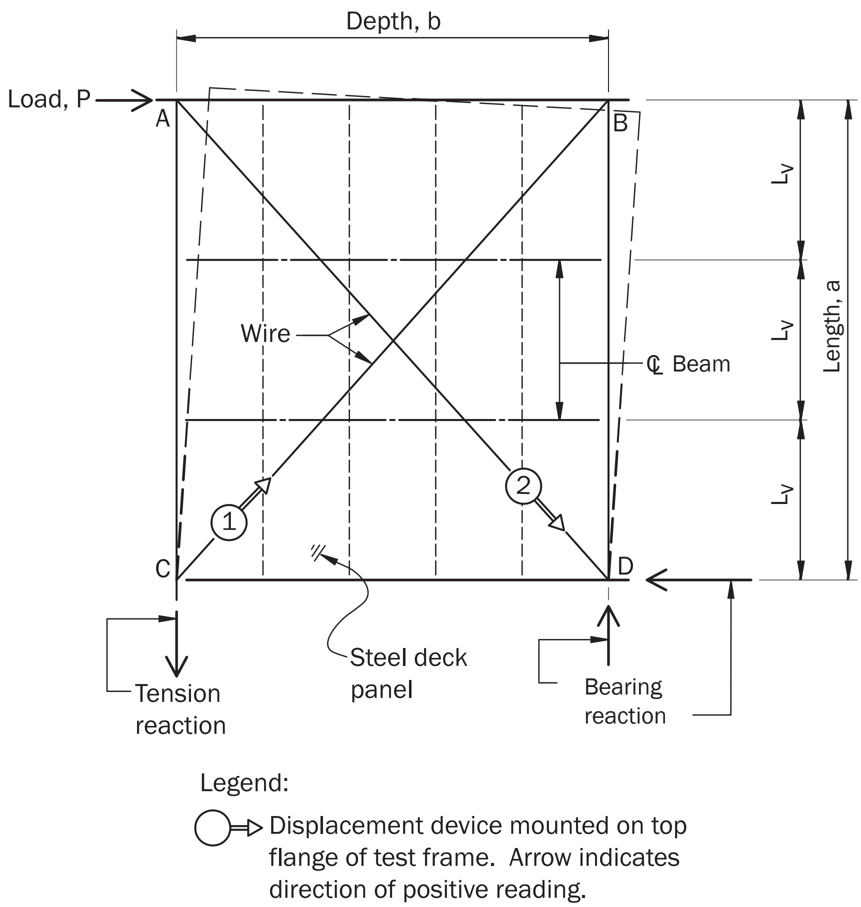


Figure 2 - Deflection Device Scheme 1

10.2 Determination of Diaphragm Web Shear Stiffness

To determine the diaphragm web shear stiffness, G′, for the full frame test assembly, the load-net deflection curve (See Figure 4) shall be plotted. When the deflection curve from no load to the initial load as described in Section 9 is inconsistent with the deflection envelope from the initial load to load, P_d, it shall be permitted to project the envelope back to the no load condition to establish the zero load deflection. That deflection shall then be used as the zero point to determine G′.

User Note:
The zero deflection adjustment offsets frame and/or deflection gage slack that is not part of the diaphragm stiffness.

The net shear deflection, Δ_n, for diagonal displacements shall be calculated in accordance with Eq. 2 (See Figure 2):

$$\Delta_n = \left(|\Delta_1| + |\Delta_2|\right)\frac{\sqrt{a^2 + b^2}}{2b} \tag{2}$$

where

$\Delta_{1,2}$ = Displacement measured at points 1 or 2. (See Figure 2)

a = Length of diaphragm test frame

b = Depth of diaphragm test frame

Alternatively, the net shear deflection, Δ_n, for rectangular displacements shall be calculated in accordance with Eq. 3 (See Figure 3):

$$\Delta_n = \Delta_3 - \left[\Delta_1 + \left(\Delta_2 + \Delta_4\right)a/b\right] \tag{3}$$

where

$\Delta_{1,2,3,4}$ = Displacement measured at points 1, 2, 3, or 4. (See Figure 3)

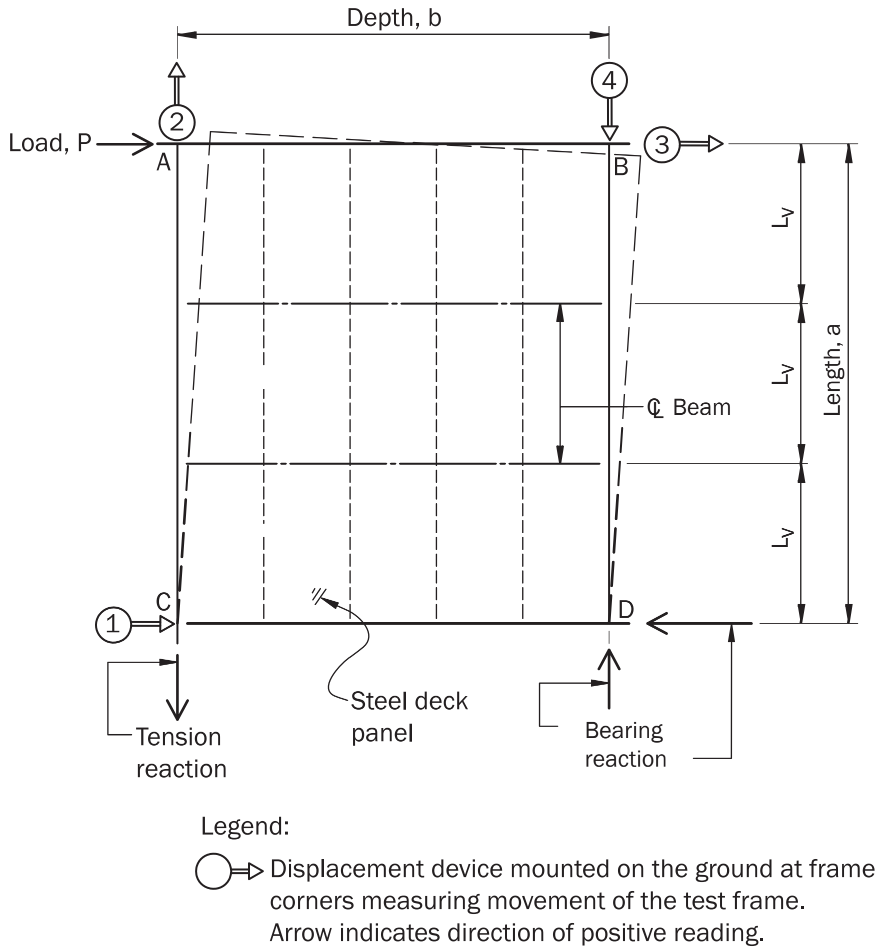


Figure 3 - Alternative Deflection Device Scheme 2

The diaphragm web shear stiffness, G′, shall be calculated as the slope of the full assembly load-net deflection curve between the curve origin and the deflection at the test load, $P_d = 0.4P_{max}$, in accordance with Eq. 4: (See Figure 4)

$$G' = \left(\frac{P_d a}{\Delta_d b}\right) \tag{4}$$

where

$$P_d = 0.4P_{max} - 0.4P_{max}\left(\frac{P_{fd}}{0.4P_{max}} - 0.02\right), \quad \text{if } \frac{P_{fd}}{0.4P_{max}} > 0.02 \tag{4a}$$

$$P_d = 0.4P_{max}, \quad \text{if } \frac{P_{fd}}{0.4P_{max}} \le 0.02 \tag{4b}$$

where

P_d = Load P at which the diaphragm stiffness shall be determined

P_{fd} = Load P from testing of bare test frame at the deflection equal to the deflection for load of $0.4P_{max}$ for stiffness

Δ_d = Net shear deflection of diaphragm test at load $0.4P_{max}$

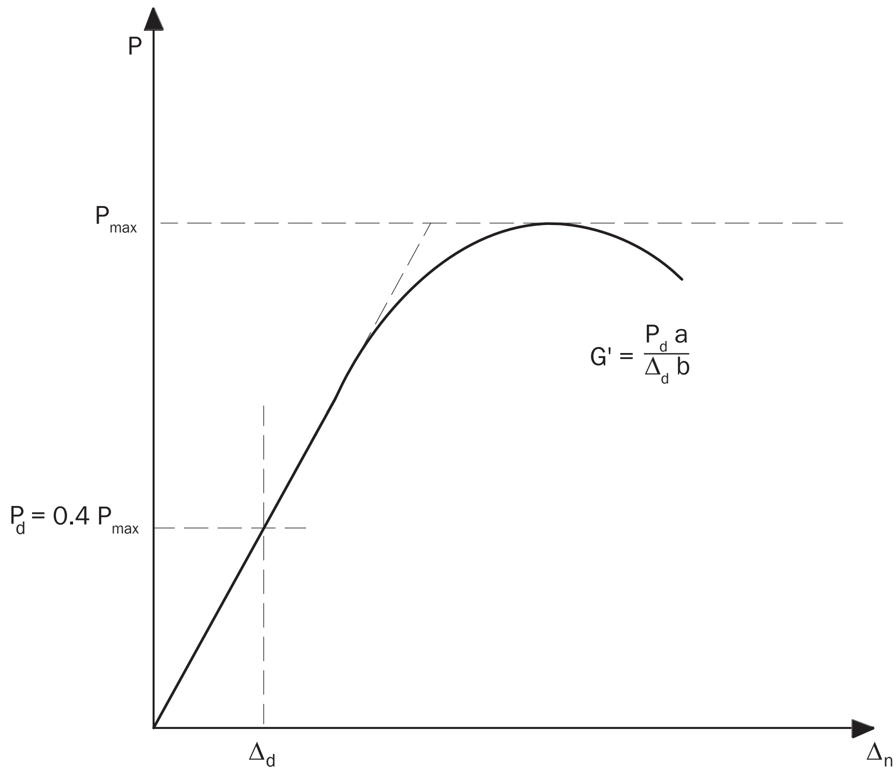


Figure 4 - Typical Load – Net Deflection Curve

10.3 Determination of Flexibility Factor

The flexibility factor of the diaphragm web, F, shall be calculated in accordance with Eq. 5:

$$F = 1/G' \tag{5}$$

11. Number of Diaphragm Tests

To confirm analytical methods for predicting diaphragm shear strength and stiffness, a minimum of three tests of a given configuration shall be required. Each test specimen shall have equal spans, the same deck thickness or one combination of thicknesses in the case of cellular decks, and one type and spacing of fasteners to end framing supports, and one type and spacing of edge fasteners to side framing. Other specimens within the same configuration shall include a variety of steel deck panel thicknesses, steel deck panel span lengths, and side seam fastener spacing within the common limits so that interpolation of results can be made.

Because the diaphragm shear stiffness, G′, varies with the span length of the steel deck panels, straight line interpolation of values of G′ with respect to steel deck panel span length shall be permitted between values of G′ of tested diaphragms.

12. Conditions of Acceptance

12.1 Evaluation of test results shall be made in conformance with Section F1.1 (a) of AISI S100 except as modified in this Standard.

12.2 The diaphragm test results shall be recorded and reduced by safety factors, Ω_d, or resistance factors, ϕ_d, as described in Section D5 of AISI S100 to obtain the allowable shear loads for allowable strength design (ASD) and design loads for load and resistance factor design (LRFD).

User Note:
Where constructions have composite or non-composite concrete fills, failures by the concrete crushing or cracking are not covered by Section D5 nor Section F1.1 of AISI S100. Usual practice has been to use a $\phi = 0.5$ or an $\Omega = 3.25$.

12.3 Equations developed from the test results shall achieve a correlation coefficient between equations and test data of 0.95 or better.

12.4 Unless only one configuration is being evaluated, at least two different configurations (i.e., fastener spacing, deck thickness) shall be tested to provide an adequate database for deriving the design equations.

12.5 No test results shall be eliminated unless a rationale for its exclusion is given.

13. Test Specimen Materials and Connectors

13.1 Steel. Steel deck panels used in diaphragm tests shall be evaluated by material property tests to determine the tensile strength, yield strength, and percent elongation in accordance with the standard for the steel grade. In addition, the base steel thickness shall be determined. Test results shall be based on the evaluation of at least three samples in each thickness. The samples shall be selected from different panels of the tested specimen.

13.2 Structural Concrete Three test cylinders of the concrete used in the diaphragm tests shall be prepared and tested in accordance ASTM C39 within 48 hours of completion of the diaphragm test.

Normal-weight aggregates in the concrete shall comply with ASTM C33.

Lightweight aggregate shall be in accordance with the requirements in ASTM C330.

13.3 Insulating Concrete The compressive strength and density of the insulating concrete used in the diaphragm tests shall be determined in accordance with ASTM C495.

For insulating concrete with aggregates, the aggregates shall conform to ASTM C332.

Insulating-cellular concrete shall be applied to the steel deck in accordance with the instructions of the manufacturer of the foaming agents. The cellular concrete mix design and placement instruction shall be included in the test report.

13.4 Welding Fillet welds, arc-spot welds, arc-seam welds, arc-plug welds, and flare-groove welds used in the diaphragm tests shall be in accordance with AWS D1.3. Headed studs shall be welded in conformance with AWS D1.1.

13.5 Mechanical Fasteners to Framing Mechanical fasteners used in the diaphragm tests shall be installed in accordance with the fastener manufacturer's recommendations. A detailed description of fasteners shall be provided, including length, diameter, thread pitch, head diameter, head shape and penetration distance into or through the substrate steel. The spacing of fasteners at ends and interior supports on different test specimens shall be varied so that the results can be interpolated. Extrapolation shall not be permitted.

For diaphragms consisting of steel deck panels with power-actuated or screw fasteners attaching the panels to steel supports, the results of the full frame assembly tests shall be adjusted when the substrate thickness or strength of the tested conditions varies from the substrate thickness and strength to be specified. The adjustments shall be based on results of tests of fasteners, connections using the fasteners and fastener application limits. Tension and shear tests of fasteners shall be in accordance with AISI S904. Shear and tension tests of connections shall be in accordance with AISI S905.

13.6 Steel Deck Panel Side Seam Mechanical Connectors Detailed descriptions of the connectors between adjacent steel deck panels used in the diaphragm tests, such as button punch or clinch connectors, shall be provided including the method of installation and the edge dimension of fasteners.

13.7 Added Elements at Steel Deck Supports Added elements used in the diaphragm test at the test frame support reactions to transfer shears and/or stiffen the deck shall be described in detail including the fasteners used to connect the elements to the steel deck and framing members.

14. Test Report

14.1 The test report shall include a description of the tested specimens, including a drawing detailing all pertinent dimensions, including base steel thickness, t, and thickness of fill, t_c.

14.2 The test report shall include the measured steel mechanical properties of the tested specimen.

14.3 Results of tests on individual materials shall be included.

14.4 The test report shall include a detailed drawing of the test setup, depicting location and direction of load application, location of displacement instrumentation and their point of reference, and details of any deviations from the test requirements stipulated in Sections 7 to 9. Additionally, photographs shall supplement the detailed drawings of the test setup. Ambient conditions at the date of construction, curing period and date and time of tests shall be reported where relevant to the performance of the tested assembly. The ambient conditions at the test site, including relative humidity, temperature and, if outside testing is performed, wind speed.

14.5 The test report shall include individual and average load-versus-deformation values and curves, as plotted directly, or as reprinted from data acquisition systems.

14.6 The test report shall include individual and average maximum test load values observed, description of the nature, type and location of failure exhibited by each specimen tested, and a description of the general behavior of the test fixture during load application. Additionally, photographs shall supplement the description of the failure mode(s).

14.7 The test report shall include a description of the test method and loading procedure used, rate of loading or rate of motion of the crosshead movement, and time to maximum load.

Commentary on AISI S907-08

Test Standard for Cantilever Test Method for Cold-Formed Steel Diaphragms

1. Scope

Shear diaphragms perform the same functions of maintaining the shape in a building roof or floor as do plate girder webs in maintaining shape between their stiffeners and flanges. However, the diaphragm stiffness, G′, usually is an order of magnitude lower than that for thin web girders.

The response of a diaphragm assembled from typical roof or floor deck panels is dependent on the panel type and thickness, panel spans, and especially on the quality of connections used. The diaphragm involves a "web" or panels, "stiffeners" or joists, purlins, and "flanges" or perimeter members. While its response may be thought of in terms of a short and deep beam, its behavior is much more related to that of truss panels having flexible diagonals.

The values of web shear strength and web shear stiffness vary based on the steel deck panel thickness and geometry, the panel width, steel framed support spacing and the methods of attachment both between the steel deck panel components and to the framing members.

The design diaphragm unit shear strength and stiffness have been developed and are available in the SDI Manual (2004 and 2006), in the MCA Primer (2004), and in ARMY TM 5-809-10 (1982) within the limits given in the documents.

7. Full Frame Test Assembly

The full frame test assembly may have perimeter members formed using various shapes including wide flange beams, C-sections, Z-sections, HSS sections, or joists. The selected shape should be sufficient to develop the required axial test forces and to permit proper perimeter connections to be made. Connections or reactions at the ends of perimeter members must be adequate to resist the developed axial forces and to transfer them to the support devices. Typical interior purlins develop only small axial forces and their end connections may be made using common clip angles. Where joists are used as interior members, it may be necessary to attach an angle or joist hanger over the joist ends or to install intermediate supports to permit proper connections at the diaphragm edge.

The study of connections on the edges parallel to the flutes of the diaphragm assembly usually is not part of a test program.

Tests on a full frame test assembly are conducted to provide information on the behavior of a complete diaphragm assembly or to provide information on a specific parameter in a given system. For studies on a complete diaphragm assembly, a minimum of five steel deck panels are required to allow reasonable force distribution across the side seams of the inner panels. Diaphragm tests conducted on less than five panels, where the edges parallel to the steel deck flutes are connected to the frame at the same spacing as the side seam connections, generally yield conservative measures of strength and stiffness. However, tests conducted with only four panels and having the edges parallel to the steel deck flutes fastened to the perimeter frame members spaced more closely than those on interior side seams, often yield artificially high strengths and stiffnesses.

The test assembly may represent a part of a larger zone in a building. The assembly should be bounded by structural members for force transfer. The frame members parallel to the jack load will have a maximum load of P_{max} and an average maximum unit shear to the diaphragm web of P_{max}/b. The frame members perpendicular to the load P will have a maximum load of P_{max} (a/b) and an average maximum unit shear to the diaphragm web of P_{max} (a/b) /a $=P_{max}/b$. Thus the average maximum unit shear to the four perimeter framing beam is constant P_{max}/b.

The test assembly should model typical constructions including perimeter transfer elements. In certain assemblies involving open web joists, the lower surfaces of the diaphragm may be above the perimeter member joist supports thus limiting perimeter attachment for the diaphragm. An edge support angle can be welded to the joist ends to accept frequent edge connections. Such angles should be attached to the joist ends and the joists then secured to the frame to permit proper shear transfer on the perimeter. Other block-like devices may be attached to the frame, between joists, to receive edge connections directly.

The ability to transfer forces across the edges of a diaphragm parallel to the deck flutes may be assessed directly through evaluation of individual connections used. For example, the shear strengths, Q_s, of headed shear studs connectors in concrete filled cold-formed steel deck diaphragms are known and studs may be placed sufficiently close so that perimeter shear failures are virtually eliminated.

End lap splices should be included in the test if they are used in the field assembly. It is however optional so that an engineer can test without the lap splices in order to isolate particular design parameters or to confirm analytical methods. Multiple layers of sheets may impact support fastener shear strength and the connection installation quality particularly in welded connections. End laps may increase stiffness. End laps should be included when they are part of the design configuration when shear strength at end laps is critical to the test evaluation.

8. Testing a Bare Frame Assembly

Testing the bare frame assembly is required to determine if the bare frame carries a significant amount of the shear in comparison to the shear being carried by the diaphragm web. If $P_{fn}/P_{max} > 0.02$ for strength analysis (Eq. 1) or $P_{fd}/(0.4P_{max}) > 0.02$ for stiffness (Eq. 4), corrections to the full frame assembly data is required.

A bare frame assembly is required to have an added weight so that the total vertical load supported by the beam, on which load P is placed, is equal to that of the same beam in the full frame assembly so that any effect of roller friction can be assessed.

9. Testing a Full Frame Assembly

The loading system used to apply loads to the full frame assembly should be calibrated conforming to the requirements of ASTM E4.

The test sequence should be long enough to allow adequate data collection but not so short to eliminate potential time dependent relaxations in the system. A ten minute minimum test time has been set but the required time usually will be longer. The spacing of each set of readings during the early part of the loading sequence when the deformations are essentially elastic should be with load increments. During the later stage of loading, particularly beyond the point where the maximum load is reached, the readings should be taken with deformation increments. The engineer in charge of the test should decide when the change should be made.

The measured stiffness of the frame support system obtained during the test normally will not be available in a field construction and are removed by the formulas indicated. In evaluation, it should be noted that the diaphragm as a "beam" is in the "short-deep beam" category. Therefore the "plane section" assumptions for longer beams do not apply. Further, the typical stiffness, G′, is an order of magnitude lower than the equivalent number for thin-web girders.

The measured stiffness, G′, reflects the performance of the diaphragm field as it resists shear forces. The frame itself usually has little resistance to movement prior to attaching the diaphragm.

The shear strength and stiffness values for diaphragms may vary in a nonlinear manner with panel length. However, reasonably conservative intermediate design values may be found by interpolation of results from similar systems of differing lengths. Extrapolation of data may lead to erroneous results.

10. Analysis of Diaphragm Tests

The text of this section provides a clarification to the analysis of test data to produce values of nominal shear strength and stiffness for use in design.

The net deflection at which stiffness or flexibility is to be determined is set at $0.4P_{max}$. The load P_d used to calculate the stiffness or flexibility of the diaphragm web is reduced if the load on the bare frame test, P_{fd}, exceeds 2 percent at that net deflection.

If the testing engineer believes the initial load deflection anomaly is partly attributable to either take up at diaphragm to panel support or side-lap connections or panel distortion, the zero point adjustment should not be made and the full displacement at $.4P_{max}$ should be used to determine G'.

11. Number of Diaphragm Tests

The test program is intended to address two cases with one being to verify a theoretical model applicable to a range of diaphragm types and the other simply to verify a very specific system. For example, test programs may be needed where a new fastener type is evaluated for its own shear strength over a range of material properties with comparisons made to some other fastener types. Theoretical methods (MCA, 2004; SDI, 2004 and 2006; and ARMY, 1982) exist for finding the effects of parameter changes on the diaphragm performance and for predicting both the nominal unit shear strength and stiffness. In such cases, the diaphragm tests themselves are considered in the mode of verifying the established model.

Other specific arrangements of panels, end closure devices, overlayments, and specific connection methods may be outside the ability of existing models for predicting response. For such cases with a very specific construction application, a minimum of three tests for each configuration is required to establish unit shear strength and stiffness.

12. Conditions of Acceptance

12.2 The safety factor, Ω_d, and resistance factor, ϕ_d, can be obtained by following AISI S100, Section F 1.1(b).

12.3 The correlation coefficient is determined by standard programs by comparing the calculated value array with the array of values from test data. The limiting value is not the deviation of an individual test from its calculated value.

12.4 If two configurations are tested, interpolated values would be acceptable but extrapolations would not be permitted.

13. Test Specimen Materials and Connectors

This section describes the materials and connectors that have been used to date. It is not the intent to limit either materials or connectors to those described. However, testing to obtain enough data to have a reasonable assurance of repeatability should be performed.

13.2 Structural Concrete: To obtain desired concrete compressive strengths, the mix of the concrete used in the diaphragm tests should follow the requirements for performing proportioning in ACI 211.1 or ACI 211.2. Test cylinders of concrete used in diaphragm test should be prepared and tested in accordance with ASTM C39.

13.4 Welding Resistance welding should conform to the requirements of AWS C1.1M/C1.1.

14. Test Report

Additional test information that would be of interest in the behavior of the test program should be reported.

Diaphragm Design References

American Iron and Steel Institute (2007), S100-07, *North American Specification for the Design of Cold-Formed Steel Structural Members*

Metal Construction Association (MCA) (2004), A Primer on Diaphragm Design, First Edition, IL.

Steel Deck Institute (SDI) (2004), SDI Diaphragm Design Manual, III

Steel Deck Institute (SDI) (2006), SDI Diaphragm Design Manual, Appendix VI

Departments of Army, Navy and Air Force (1982), TM 5-809-10 Seismic Design for Buildings

ASTM International (ASTM) (2007), ASTM E4-07 Standard Practice for Force Verification of Testing Machines

ASTM International (ASTM) (2005e1), ASTM C39-05e1 Standard Practice for Making and Curing Concrete Test Specimens in the Field

American Concrete Institute (Reapproved 2002), ACI 211.1-91 Standard Practice for Selecting Proportions for Normal, Heavyweight and Mass Concrete

American Concrete Institute (Reapproved 2004), ACI 211.2-98 Standard Practice for Selecting Proportions for Structural Lightweight Concrete

American Welding Society (2000), AWS C1.1M/C1.1-2000 (R2006) Recommended Practices for Resistance Welding

AISI S908-08

Base Test Method for Purlins Supporting a Standing Seam Roof System

1. Scope

1.1 The purpose of this test is to obtain the reduction factor for use in determining the nominal flexural strength of a purlin supporting a standing seam roof system.

1.2 This test method, herein also referred as "Base Test Method", applies to an assembly consisting of the standing seam panel, purlin, and attachment devices used in the system being tested. The test specimen boundary conditions, described in Section 7.6, apply only to standing seam roof systems for which the roof deck is positively anchored to the supporting structural system at one or more purlin or eave member lines.

1.3 The Base Test Method is used to evaluate the nominal flexural strength of C- and Z-sections of multi-span, multiple purlin line, standing seam systems, with or without discrete intermediate braces.

1.4 The Base Test Method is applicable to both "rib" or "pan" type standing seam roof panels with "sliding" or "fixed" type clips.

1.5 The Base Test Method is conducted using standing seam roof panels, clips, fasteners, insulation, thermal blocks, discrete braces, and purlins as used in the actual standing seam roof system except as noted in Section 1.6.

1.6 Tests conducted with insulation are applicable to identical systems with thinner or no insulation.

2. Referenced Documents

The following documents or portions thereof are referenced within this Standard and shall be considered as part of the requirements of this document.

a. American Iron and Steel Institute (AISI), Washington, DC:

S100-07, *North American Specification for the Design of Cold-Formed Steel Structural Members*

b. ASTM International (ASTM), West Conshohocken, PA:

A370-07b, Standard Test Methods and Definitions for Mechanical Testing of Steel Products

E6-07b, Standard Terminology Relating to Methods of Mechanical Testing

IEEE/ASTM-SI-10-02, American National Standard for Use of the International System of Units (SI): The Modern Metric System

3. Terminology

Where the following terms appear in this Standard they shall have the meaning as defined herein. Terms not defined in Section 3 of this Standard or AISI S100, ASTM E6, or IEEE/ASTM-SI-10 shall have the ordinary accepted meaning for the context for which they are intended.

Failure. A stage at which the specimen will accept no further loading.

Fixed clip. A hold down clip which does not allow the roof panel to move independently of the roof substructure.

Insulation. Glass fiber blanket or rigid board.

Lateral. A direction normal to the span of the purlins in the plane of the roof sheets.

Thermal block. Strips of rigid insulation located directly over the purlin between clips.

Negative moment. A moment which causes tension in the purlin flange attached to the clips and standing seam panels.

Pan type standing seam roof. A "U" shaped panel which has vertical sides.

Positive moment. A moment which causes compression in the purlin flange attached to the clips and standing seam panels.

Rib type standing seam roof. A panel which has ribs with sloping sides and forms a trapezoidal shaped void at the side lap.

Sliding clip. A hold down clip which allows the roof panel to move independently of the roof substructure.

Standing seam roof system. A roof system in which the side laps between the roof panels are arranged in a vertical position above the roof line. The roof panel system is secured to the purlins by means of concealed hold down clips that are attached to the purlins with mechanical fasteners.

Yield Stress. Generic term to denote either yield point or yield strength, as appropriate for the material.

4. Symbols:

b	= Flange width of the purlin
d	= Depth of the purlin
B	= Purlin spacing
F_y	= Design yield stress
F_{yt}	= Measured yield stress of tested purlin
I_x	= Moment of inertia of full unreduced section about x-axis
I_{xy}	= Product of inertia of flange about major axis
L	= Span of the purlins tested, center to center of the supports
M_n	= Nominal flexural strength of a fully constrained beam, S_eF_y
$\bar{M}_{nt_{min}}$	= Average flexural strength of the thinnest sections tested
$\bar{M}_{nt_{max}}$	= Average flexural strength of the thickest sections tested
M_{nt}	= Flexural strength of a tested purlin, $S_{et}F_{yt}$
M_{ts}	= Failure moment for the single span purlins tested, $w_{ts}L^2/8$
p_d	= Weight of the specimen (force/area)
p_{ts}	= Failure load (force/area) of the single span system tested
P_L	= Lateral anchorage force in accordance with Section D6.3.1 of the AISI S100
R_t	= Modification factor from test, M_{ts}/M_{nt}
R	= Reduction factor computed for nominal purlin properties
R_{tmin}	= Mean minus one standard deviation of the modification factors of the three thinnest purlins tested

$R_{t_{max}}$ = Mean minus one standard deviation of the modification factors of the three thickest purlins tested
s = Tributary width of the purlins tested
S_e = Section modulus of the effective section
S_{et} = Section modulus of the effective section of the tested member using measured dimensions and the measured yield stress
t = Purlin thickness
w_{ts} = Failure load (force/length) of the single span purlins tested

5. Units of Symbols and Terms

Any compatible system of measurement units shall be permitted to be used in this Standard, except where explicitly stated otherwise. The unit systems considered in this Standard shall include U.S. customary units (force in kips and length in inches) and SI units (force in Newtons and length in millimeters) in accordance with IEEE/ASTM-SI-10.

6. Apparatus

6.1 The testing apparatus shall consist of a test chamber capable of supporting a positive or negative internal pressure differential. A rectangular frame shall be constructed of any material with the strength and rigidity to provide the desired pressure differential without collapse. See Figure 1 for a typical test chamber. Other chamber orientations shall be permitted.

6.2 The length of the chamber shall be determined by the maximum length of the purlins as required by Section 8.2. The width of the chamber shall be determined by the maximum panel length as required by Section 7.9. Allowance shall be made in the interior chamber dimensions to accommodate structural supports for the purlins and sufficient clearance on all sides to prevent interference of the chamber wall with the test specimen as it deflects.

6.3 The height of the chamber shall permit assembly of the specimen and to insure adequate clearance at the maximum deflection of the specimen.

6.4 The chamber shall be sealed in a manner to prevent air leakage. All load carrying elements of the specimen or its supports shall transfer the load to the frame support; the specimen, including intermediate brace, shall not be attached to the chamber in any manner that would impede the deflection of the specimen.

6.5 The test chamber shall be sealed against air leakage by applying 6 mil (0.15 mm) maximum thickness polyethylene sheets, large enough to accommodate the system configuration and deflections. The polyethylene shall be located on the high pressure side of the panel with sufficient folds so as not to inhibit the spread of panel ribs under load. Edges of the polyethylene sheets shall be sealed against air leakage with tape or other suitable methods. Polyethylene sheets around the perimeter of the specimen shall be draped so as not to impede deflection or deformation of the specimen.

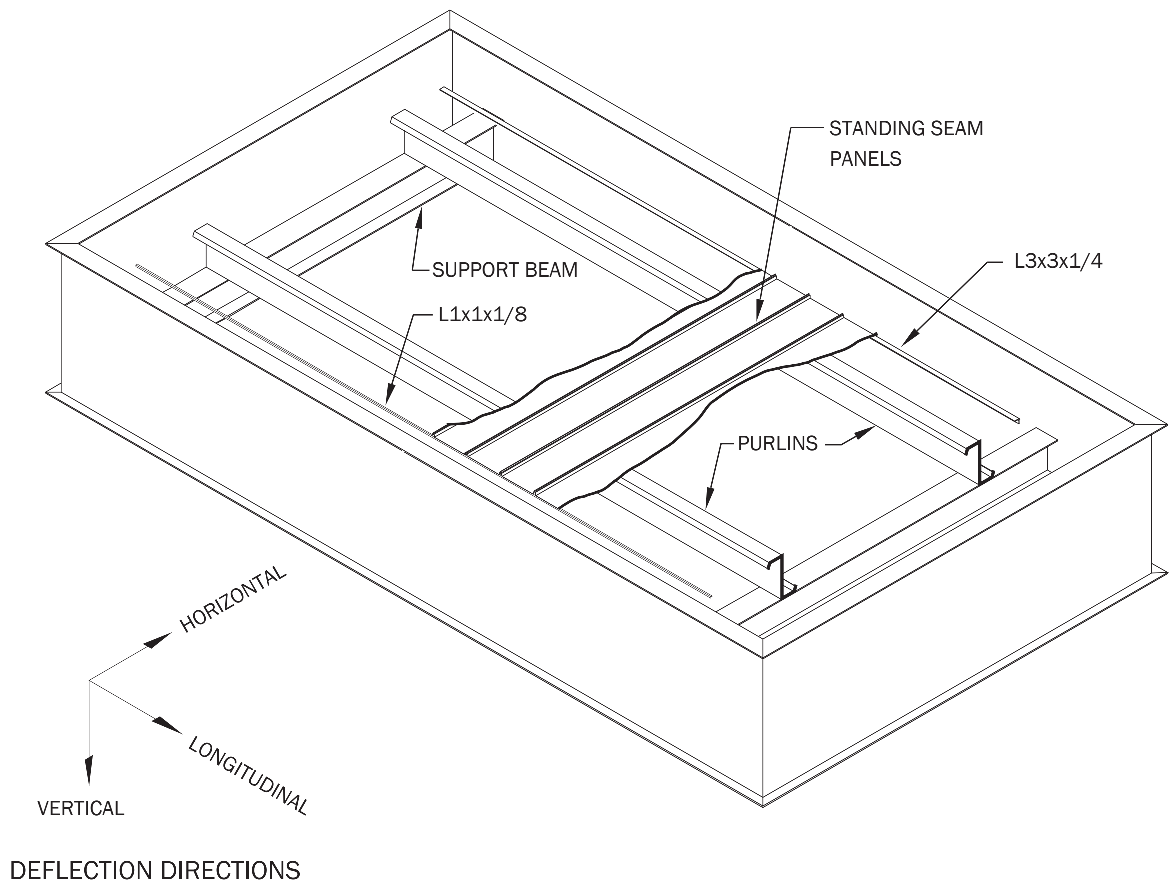


Figure 1 – Test Chamber

6.6 When a specimen smaller than the test chamber is tested, other panels and structure shall be installed to complete the coverage of the chamber opening. No attachment shall be made between the test specimen and this supplemental coverage.

6.7 An air pump shall be used to create the pressure differential in the chamber. The pump shall be of enough capacity to reach the required test values by the applicable specifications.

6.8 The type of air pump being used shall determine the method of control. This control shall be able to regulate the pressure differential in the chamber to ± 1 psf (0.05 kPa). This shall be accomplished by (a) a variable speed motor on the pump, (b) valving on the pump, or (c) variable size orifices on the chamber. It shall be permitted to use multiple pumps where very large chambers are being used. One pump connection to the chamber shall be satisfactory.

6.9 A minimum of two pressure differential measuring devices shall be monitored throughout the duration of the test. These devices shall be capable of measuring the pressure differential to ± 1 psf (0.05 kPa).

7. Test Specimens

7.1 Test purlins shall be supported at each end by a steel beam. The beams shall be simply supported and one of the frame end beams shall be free to translate laterally to relieve any longitudinal catenary forces in the specimen. Purlins shall be connected to the supporting beams as recommended in the field erection drawings. Figure 1 shows the directional axes that are referred to in this test procedure.

7.2 Panel supporting clips, fasteners, and panels shall be installed as recommended in the field erection drawings.

7.3 Means of providing restraint of purlins at the support shall be as required for use in actual field application, and shall be installed as recommended on the field erection drawings.

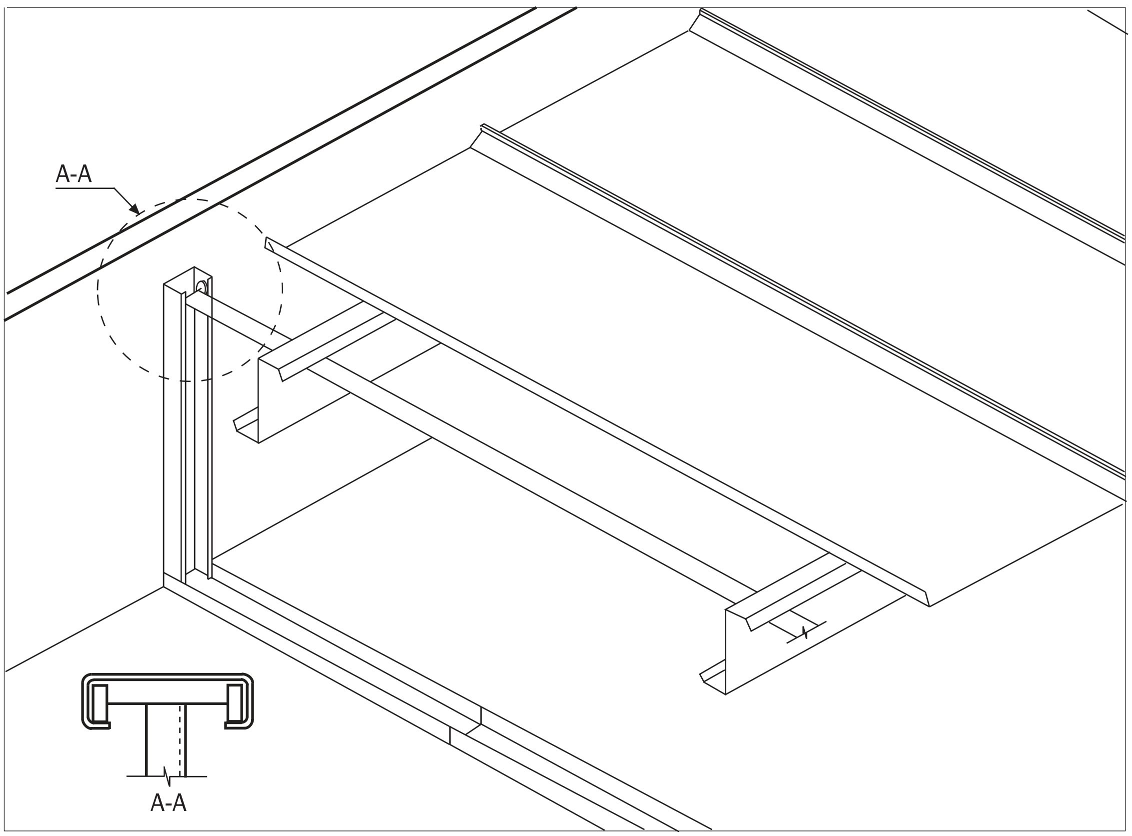


Figure 2 - Example of Lateral Restraint Test Device

7.4 The purlins shall be arranged either with their flanges facing in the same direction or with their flanges opposed. If the test is performed with the purlin flanges opposed, they shall be field installed with their flanges opposed.

7.5 For tests including intermediate discrete point braces, the braces used in the test shall be installed in such a manner so as not to impede the vertical deflection of the specimen.

User Note:
Figure 2 shows one means of satisfying this requirement.

7.6 A 1 in. x 1 in. (25 mm x 25 mm) continuous angle with a maximum thickness of 1/8 in. (3 mm) or a member of compatible stiffness shall be attached to the underside at each end of the panels to prevent separation of the panels at the ends of the seam. Fasteners shall be placed on both sides of each major rib. If the specimen is arranged with the purlin flanges facing in the same direction, a 3 in. x 3 in. (76 mm x 76 mm) continuous angle with a maximum thickness of 1/4 in. (6 mm) or a member of compatible stiffness shall be permitted to be substituted for the 1 in. x 1 in. (25 mm x 25 mm) angle at the end of the panel, corresponding to the eave of the building using the standard panel to eave fastening system. See Figure 1.

7.7 All transverse panel ends shall be left free to displace vertically under load. When the 3 in. x 3 in. (76 mm x 76 mm) eave angle is used and the purlin flanges face in the same direction, the eave angle shall be permitted to be restrained against horizontal deflection at its ends as shown in Figure 1, providing vertical deflection is left unrestrained.

7.8 Panel joints shall not be taped and no tape shall be used to restrict panel movement.

7.9 Panel length to be used in the test shall be, as a minimum, that length which provides full engagement of the panel to purlin clip and attachment of the 1 in. x 1 in. (25 mm x 25 mm) angle at the panel ends; but a length not greater than that required to achieve zero slope of the panel at the purlin support.

7.10 The spacing of purlins being tested shall not exceed the spacing practically used with the roof system. Results from this test shall be permitted to be used in designing purlins of the same profile that are spaced closer together than the spacing used in the tests.

8. Test Procedure

8.1 A test series shall be conducted for each purlin profile, specified steel grade, and each panel system. Any variation in the characteristics or dimensions of panel or clip shall constitute a change in panel system. The thickness of insulation used in the test shall be applicable to identical systems with thinner or no insulation. Any change in purlin shape or dimension other than thickness shall constitute a change in profile. However, the lip dimension shall be permitted to vary with section thickness consistent with the member design and not constitute a change in profile.

8.2 No fewer than six tests shall be run for each combination of purlin profile and panel system. Three tests shall be conducted with the thinnest purlin of the profile and three tests shall be conducted with the thickest purlin of the profile. All tests shall be conducted using the same purlin span which is the same or greater than the span used in actual field conditions.

8.3 If an inventory consists of different clip types, a specific purlin depth and profile but with different flange widths, and identical panel profiles except for thickness, a rational procedure shall be permitted to be used to reduce the number of required tests.

8.4 The physical and material properties shall be determined in accordance with ASTM A370 using coupons taken from the web area of the failed purlin. Coupons shall not be taken from areas where cold-working stresses could affect the results.

8.5 For gravity loading, a pressure differential load shall be applied to the system to produce a positive moment in the system. For uplift loading, a pressure differential load shall be applied to the system to produce a negative moment in the system.

8.6 An initial load equal to 5 psf (0.25 kPa) differential pressure in the direction of the test load shall be applied and removed to set the zero readings before actual system loading begins.

8.7 The system shall be loaded to failure and the mode of failure noted. The pressure differential at which the system fails shall be recorded as the failure load of the specimen. When the test must be stopped due to a flexural failure of the panel or web crippling of the purlin, it shall be permitted to exclude the test from the test program.

8.8 Vertical deflection measurements shall be taken at the mid-span of both purlins. The deck deflection in the horizontal direction shall be measured at the seam joint nearest the center of the test specimen.

8.9 Deflections and pressures shall be recorded at pressure intervals equal to a maximum of 20 percent of the anticipated failure load.

9. Test Evaluation

9.1 The single span failure load shall be obtained from the Base Test where a uniform load is applied until failure occurs. The computation of the failure load, w_{ts}, shall be dependent on the purlin orientation for Z-purlins and on the nature of the load as follows:

1. For Z-purlins tested for gravity loading, with flanges facing the same direction and with the top flanges of the purlins not restrained by anchorage to a point external to the panel/purlin system, the failure load shall be calculated as follows:

$$w_{ts} = (p_{ts} + p_d)s + 2P_L\left(\frac{d}{B}\right) \quad (1)$$

where

$$P_L = 0.5\left(\frac{C2}{1000}\frac{I_{xy}L}{I_x d} + C3\frac{0.25bt}{d^2}\right)(p_{ts} + p_d)s \quad (2)$$

	C2	C3
Standing Seam	8.3	28

2. For Z-purlins tested for gravity loading with flanges opposed and for C-sections tested for gravity loading, the failure load shall be determined as follows:

$$w_{ts} = (p_{ts} + p_d)s \quad (3)$$

3. For Z-purlins or C-sections tested for uplift loading, the failure load shall be determined as follows:

$$w_{ts} = (p_{ts} - p_d)s \quad (4)$$

The expression $2P_L(d/B)$ in Equation (1) takes into account the effect of the overturning moment on the system due to the anchorage forces, as defined in Section D6.3.1 of AISI S100, applied at the top flange of the purlin by the panel and resisted at the bottom flange of the purlin at the support. The expression $2P_L(d/B)$ shall be applied only to Z-sections under gravity loading when the purlin flanges are facing in the same direction, but shall not be included in those systems where discrete point braces are used when the braces are restrained from lateral movement. The expression $2P_L(d/B)$ shall not be applied unless the downhill purlin is the first to fail.

9.2 From the single span failure load, w_{ts}, the maximum single span failure moment M_{ts} shall be calculated as follows:

$$M_{ts} = w_{ts} L^2 / 8 \tag{5}$$

9.3 The single span Base Test moment shall be the maximum moment that the system can resist with the purlin size used in the test. The maximum allowable moment of a roof system purlin, simple span or continuous, shall be limited by the results of this test. The gravity load results shall apply for positive moment regions in the span and uplift load results shall apply for negative moment regions in the span.

9.4 Using Section C3.1.1(a) of AISI S100, the flexural strength of each tested purlin, M_{nt}, of a fully constrained beam shall be calculated as follows:

$$M_{nt} = S_{et} \times F_{yt} \tag{6}$$

where S_{et} is the section modulus of the effective section calculated using the measured cross-sectional dimensions and measured yield stress and F_{yt} is the measured yield stress obtained in accordance with Section 9.4.

9.5 The modification factor, R_t, shall be calculated for each purlin tested as:

$$R_t = M_{ts} / M_{nt} \tag{7}$$

9.6 For purlins of the same profile, specified steel grade, and panel system as tested, the reduction factor shall be calculated as follows:

$$R = \left(\frac{R_{tmax} - R_{tmin}}{\bar{M}_{nt_{max}} - \bar{M}_{nt_{min}}}\right)\left(M_n - \bar{M}_{nt_{min}}\right) + R_{tmin} \le 1.0 \tag{8}$$

where

R_{tmin} = Mean minus one standard deviation of the modification factors of the three thinnest purlins tested, calculated in accordance with Section 9.5. This value may be greater than 1.0

R_{tmax} = Mean minus one standard deviation of the modification factors of the three thickest purlins tested, calculated in accordance with Section 9.5. This value may be greater than 1.0

M_n = Nominal flexural strength of section for which R is being evaluated (S_eF_y)

$\bar{M}_{nt_{min}}$ = Average flexural strength of the thinnest section tested, calculated in accordance with Section 9.4

$\bar{M}_{nt_{max}}$ = Average flexural strength of the thickest section tested, calculated in accordance with Section 9.4

9.7 If the test is performed with the purlins opposed or with an eave member at one or more edges, the diaphragm strength and stiffness of the panel system shall be tested unless the purlins are also opposed in actual field usage. The anchorage forces for the system braced in the manner tested shall be calculated in accordance with Section D6.3.1 of AISI S100. The diaphragm strength of the panel system shall be equal to or greater than the calculated brace force at the failure load of the purlin. The stiffness of the diaphragm shall be such that the deflection of the diaphragm is equal to or less than the purlin span divided by 360 when subjected to the calculated brace force at the failure load of the purlin.

10. Test Report

10.1 Documentation - The report shall include who performed the test and a brief description of the system being tested.

10.2 The documentation shall include test details with a drawing showing the test fixture and indicating the components and their locations. A written description of the test setup detailing the basic concept, loadings, measurements, and assembly shall be included.

10.3 The report shall include a drawing showing the actual geometry of all specimens including material specifications and test results defining the actual material properties - material thickness, yield stress, tensile strength, and percent elongation.

10.4 The report shall include the test designation, loading increments, displacements, mode of failure, failure load, and specimen included for each test.

10.5 The report shall include a description summarizing the test program results to include specimen type, span, failure moments for the test series, and the supporting calculations.

Commentary on AISI S908-08

Base Test Method for Purlins Supporting a Standing Seam Roof System

This test method provides the requirements for evaluating the resisting moment for cold-formed C- and Z-sections used with standing seam roof systems. The resisting moment is the multiplication of a reduction factor and the moment resistance of the corresponding member under the fully braced condition. The reduction factor reflects the ability of a particular standing seam roof system to provide lateral and rotational support to the purlins to which it is attached. This applies to discrete lateral and torsional bracing when the sheeted flange of the purlin is the compression flange, as in gravity loading cases, and when the unsheeted flange is the compression flange, as in wind uplift cases.

Due to the many different types and construction of standing seam roof systems and their attachments, it is not practical to develop a generic method to predict the interaction of a particular standing seam roof system and supporting structure. Therefore, the amount of resisting moment which the supporting purlins can achieve can vary from the fully braced condition to the unbraced condition for a given system.

The test method, herein referred to as the "Base Test Method", is the result of extensive testing of various combinations of purlins, standing seam panels, and fastening devices. The tests were conducted over several years, benefiting from the experience provided by technical and industry experts. This procedure utilizes the results obtained from single span tests to predict the strength of multi-span conditions. The validity of this test method has been established by a research program at Virginia Polytechnic Institute and State University and documented in References 3 through 8.

In Standard Section 8.3, a rational procedure is permitted to be used to reduce the number of required tests. Such a rational procedure has been provided in reference 8.

Due to anchorage design revisions in the 2007 edition of AISI S100 (AISI, 2007), Equation (2) in S908 was revised accordingly. Equation (2) is the simply supported case of *Specification* Equation D6.3.1-2 with $\theta = 0$.

References

(1) American Iron and Steel Institute (2007), *North American Specification for the Design of Cold-Formed Steel Structural Members*, 2007.

(2) AISI Task Committee on Base Test and Anchorage Questions, "Frequently Asked Questions Concerning the AISI Base Test Method and the uses of the AISI Anchorage Equations", CCFSS Technical Bulletin, Vol. 12, No. 1 , University of Missouri – Rolla, Rolla, Missouri, 2003

(3) S. Brooks and T. Murray, "Evaluation of the Base Test Method for Predicting the Flexural Strength of Standing Seam Roof Systems under Gravity Loading," MBMA Project 403, VPI Report No. CE/VPI-ST89/07, Metal Building Manufacturers Association, 1300 Sumner Ave., Cleveland, Ohio 44115, July 1989, Revised November 1990.

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(6) B. Anderson and T. Murray, “Base Test Method for Standing Seam Roof Systems Subject to Uplift Loading - Phase I,” MBMA Project 501, VPI Report No. CE/VPI-ST90/06, Metal Building Manufacturers Association, 1300 Sumner Ave., Cleveland, Ohio 44115, December 1990, Revised December 1991.

(7) A. Pugh and T. Murray, “Base Test Method for Standing Seam Roof Systems Subject to Uplift Loading - Phase II,” MBMA Project 602, VPI Report No. CE/VPI-ST91/17, Metal Building Manufacturers Association, 1300 Sumner Ave., Cleveland, Ohio 44115, December 1991.

(8) T. Murray, “Base Test Method for Uplift Loading - Final Report,” MBMA Project 501, 602 and 702, VPI Report No. CE/VPI-ST-97/10, Metal Building Manufacturers Association, 1300 Sumner Ave., Cleveland, Ohio 44115, November 1997.

(9) Trout, A. M. and T. M. Murray, “Reduced Number of Base Tests,” Research Report CE/VPI-ST00/17 submitted to Metal Building Manufacturers Association, 1300 Sumner Ave., Cleveland, Ohio, 44115, December 2000.

AISI S909-08

Standard Test Method for Determining the Web Crippling Strength of Cold-Formed Steel Beams

1. Scope

1.1 This performance test method establishes procedures for conducting tests to determine the web crippling strength of cold-formed steel flexural members.

1.2 This Standard test method describes the procedure for determining the following web crippling strengths:

Interior-One-Flange Loading (IOF) (Figure 1)

End-One-Flange Loading (EOF) (Figure 2)

Interior-Two-Flange Loading (ITF) (Figure 3)

End-Two-Flange Loading (ETF) (Figure 4)

1.3 The test method is applicable to single-web, multiple-web and built-up web sections. See Figures 5, 6 and 7, respectively.

1.4 This Standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user to establish appropriate safety and health practices, and determine the applicability of regulatory limitations prior to use.

2. Referenced Documents

The following documents or portions thereof are referenced within this Standard and shall be considered as part of the requirements of this document.

a. American Iron and Steel Institute (AISI), Washington, DC:

S100-07, *North American Specification for the Design of Cold-Formed Steel Structural Members*

b. ASTM International (ASTM), West Conshohocken, PA:

A370-07b, Standard Test Methods and Definitions for Mechanical Testing of Steel Products

E6-07b, Standard Terminology Relating to Methods of Mechanical Testing

IEEE/ASTM-SI-10-02, American National Standard for Use of the International System of Units (SI): The Modern Metric System

3. Terminology

Where the following terms appear in this Standard they shall have the meaning as defined herein. Terms not defined in Section 3 of this Standard, AISI S100 or ASTM E6 shall have the ordinary accepted meaning for the context for which they are intended.

Interior-one-flange loading and reaction (IOF). A condition where the distance from the edge of the bearing to the end of the member is greater than 1.5h, and the clear distance between the bearing edges of adjacent opposite concentrated loads or reactions is equal to or greater than 1.5h. See Figure 1.

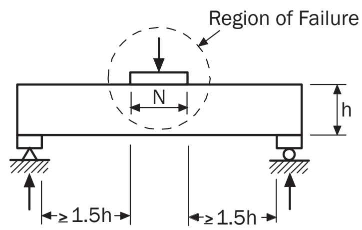

Figure 1 – Interior-One-Flange Loading (IOF)

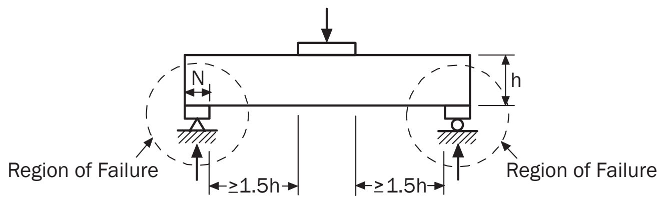

Figure 2 – End-One-Flange Loading (EOF)

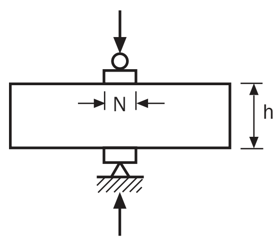

Figure 3 – Interior-Two-Flange Loading (ITF)

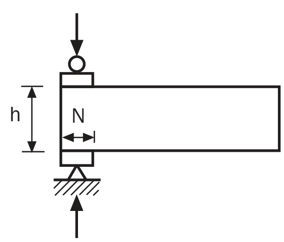

Figure 4 – End-Two-Flange Loading (ETF)

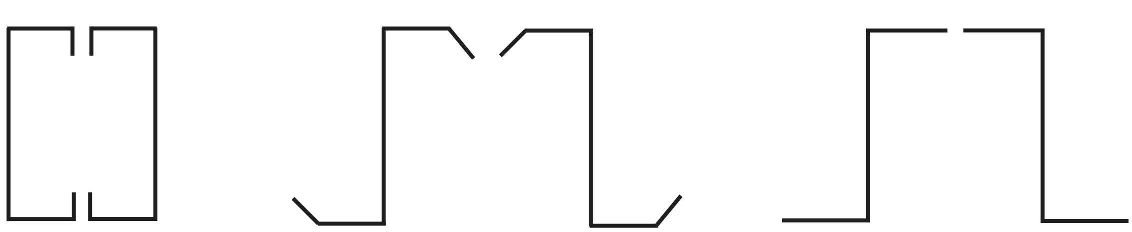

Figure 5 – Single-Web Cross-Sections

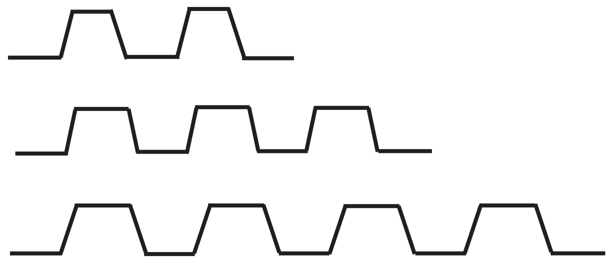

Figure 6 – Multi-Web Cross-Sections

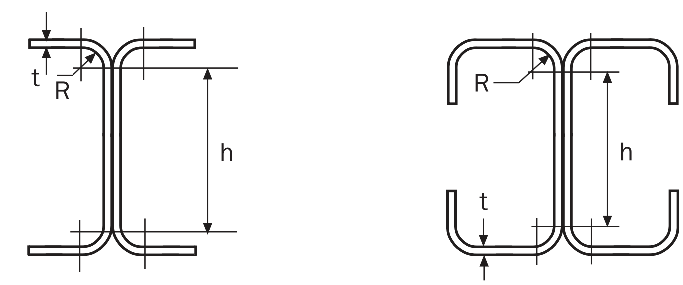

Figure 7 – Built-Up Cross Sections

End-one-flange loading and reaction (EOF). A condition where the distance from the edge of the bearing to the end of the member is equal to or less than 1.5h, and the clear distance between the bearing edges of adjacent opposite concentrated loads or reactions is equal to or greater than 1.5h. See Figure 2.

Interior-Two-Flange Loading (ITF). A condition where the distance from the edge of the bearing to the end of the member is greater than 1.5h, except as otherwise noted, and the clear distance between the bearing edges of adjacent opposite concentrated loads or reactions is equal to or less than 1.5h. See Figure 3.

End-Two-Flange Loading (ETF). A condition where the distance from the edge of the bearing to the end of the member is equal to or less than 1.5h, and the clear distance between the bearing edges of adjacent opposite concentrated loads or reactions is equal to or less than 1.5h. See Figure 4.

4. Symbols

h = Depth of flat portion of web element measured along plane of web
L_{min} = Minimum length of test specimen
N = Length of bearing
t = Web thickness
R = Inside bend radius
θ = Angle between plane of web and plane of bearing surface

5. Units of Symbols and Terms

Any compatible system of measurement units shall be permitted to be used in this Standard, except where explicitly stated otherwise. The unit systems considered in this Standard shall include U.S. customary units (force in kips and length in inches) and SI units (force in Newtons and length in millimeters) in accordance with IEEE/ASTM-SI-10.

6. Apparatus

6.1 In lieu of using a test machine, the load shall be permitted to be applied by a hydraulic or pneumatic cylinder. When a cylinder is used, a calibrated load cell shall be used to measure the applied load to within ± 2 percent.

User Note:
The test method is generally suitable for either hydraulic or screw operated testing machines that comply with the requirements of ASTM E4-07, Standard Practices for Force Verification of Testing Machines be used as applicable.

7. Test Specimen

7.1 The test specimen shall be both laterally and torsionally stable. Thus, for a geometry that does not permit the application of the load through the shear center, (e.g. a C-shape), or for a geometry having oblique principal axes (e.g. Z-shape), the test specimen shall consist of two opposed sections as shown in Figure 5. Alternatively, the shapes shall be permitted to be positioned to represent in-place conditions with appropriate lateral stability provided (e.g. C-shapes facing the same direction with the flanges attached by sheathing).

7.2 When evaluating the web crippling strength of a specific single-web cross section, the test specimen as described in Section 7.1 shall be constructed with two cross sections of like geometry, dimensions, and material properties. For consideration of a specific structural condition, the in-place condition shall be simulated by the test specimen (e.g. a lapped section at the interior support).

7.3 The cold-formed steel shapes shall be interconnected using rigid connecting elements, (e.g. L3/4x3/4x1/8 in inches and L20x20x3 in mm) to form the box shape. Connecting elements of equivalent stiffness shall be permitted (e.g. sheathing). The width of the connecting elements shall not be so large that they influence the web crippling deformation.

7.4 The rigid connecting elements shall be connected to the top flange of the cold-formed steel shape using screws or bolts. It shall be permitted to attach the rigid connecting elements to the bottom flange as well.

User Note:
A self-drilling screw is commonly used.

7.5 When using rigid connecting elements, they shall be located at approximately the 1/4 and 3/4 points along the longitudinal axis of the box shape.

7.6 For built-up shapes, (e.g. back-to-back C-shapes or nested Z-shapes), the fastener type and pattern used to fabricate the shape shall replicate the in-place condition.

7.7 For sections that may experience a spreading of their webs when under loads, such as a hat section, the open side of the cross section shall be permitted to be laterally restrained by rigid elements as defined in Sections 7.3 and 7.4 assuming in-place conditions are reflected.

7.8 The length of the test specimen shall be defined based on the loading condition and the in-place conditions. Conservatively, it shall be permitted to use the following minimum specimen lengths:

IOF Loading: L_{min} = 3.0 h + bearing plate lengths (See Figure 1)

EOF Loading: L_{min} = 3.0 h + bearing plate lengths (See Figure 2)

ITF Loading: L_{min} = 3.0 h (See Figure 3)

ETF Loading: L_{min} = 3.0 h (See Figure 4)

where h = depth of the flat portion of the web measured along the plane of the web.

User Note:
For ITF loading, L_{min} = 3.0 h provides a conservative web crippling strength. Based on in-place conditions, it should be permitted to use a longer length, for example L = 5.0 h. However, longer lengths for IOF loading are not recommended since it may result in premature failure resulting from combined bending and web crippling.

7.9 The length of the bearing plate, N, shall replicate in-place conditions.

7.10 The cold-formed steel shape shall be connected to its support member replicating in-place conditions. For conservative results, it shall be permitted to omit the support connection.

7.11 For conservative results, a simply supported condition shall be permitted. Alternatively, the support condition shall replicate the in-place conditions (e.g. C-shapes nested into a track section).

7.12 It shall be permitted to use the test specimen configuration and bracing that replicate the in-place conditions.

8. Test Procedure

8.1 A test series shall consider each steel grade and cross-section geometry.

8.2 A test series shall consist of no fewer than three tests for each unique cross section geometry and steel grade. The safety factor or resistance factor used in design shall be computed in accordance with Section F1 of AISI S100.

8.3 The physical and material properties of the sheet steel shall be determined in accordance with ASTM A370. The coupons shall be taken from flat sheet cut from the coil used to fabricate the cold-formed steel shapes, or from the web element of the shape. Coupons shall be taken from areas where cold-working stresses will not affect the results.

8.4 The test specimen shall be loaded to failure and the mode of failure shall be noted. Failure shall be considered as at the point at which the specimen will accept no further load. For those members or assemblies that fail in a progressive manner (e.g. a mechanism whereby there is an initial web crippling failure followed by a change in the specimen configuration and then continued increase in load carrying capacity), the failure load shall be permitted to be taken as the first local maxima in the load deflection curve.

9. Test Evaluation

9.1 The measured failure load per web at the location of failure shall be computed using measured values, common methods of statics or other structural analysis methods as required.

10. Test Report

10.1 The objectives and purposes of the test series shall be stated at the outset of the report so that the necessary test results such as the failure load and the mode of failure are identified.

10.2 The type of tests, the testing organization, the supervising engineer, and the dates on which the tests were conducted shall be included in the documentation.

10.3 The test specimen shall be fully documented, including:

(a) the measured dimensions, material properties and identification data of each specimen:
material thickness, yield stress, percent elongation, cross-section dimensions, length of bearing plate(s), specimen length, support conditions, manufacturer, any other distinguishing characteristics.
(b) location of additional stiffeners,
(c) location of lateral braces,
(d) location and size of connecting elements, and
(e) type and size of fasteners used.

10.4 The report shall include the type of testing machine, loading increments, and supports. If a hydraulic cylinder and load cell are used, they shall be described. The last date of calibration for the test machine or load cell shall be recorded.

10.5 The report shall include a description summarizing the test program results including the specimen type, span length, failure loads for the test series, representative load-deflection curves, and supporting calculations.

Commentary:
This test method is developed based on the research report by LaBoube and Schuster (2002), Standard Test Method for Determining the Web Crippling Strength of Cold-Formed Steel Members, published by American Iron and Steel Institute.

AISI S910-08

Test Method for Distortional Buckling of Cold-Formed Steel Hat Shaped Compression Members

1. Scope

1.1 This test method establishes procedures for determining the distortional buckling strength of cold-formed steel hat shaped compression members with a hat shaped cross section.

1.2 This Standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this Standard to establish appropriate safety and health practices, and determine the applicability of regulatory limitations to use.

Commentary:
Distortional buckling involves both rotation of the compression element as well as translation of the compression element about fold lines. Distortional buckling reduces the axial load-carrying capacity that would otherwise be limited by general yielding, local buckling, or overall column buckling. AISI S100 can be used to determine the column buckling strength limited by general yielding, local, distortional, and overall buckling. The Direct Strength Design included in Appendix A of AISI S100 provides an alternative design approach for determining member strengths. The AISI S902 can be used to determine the column capacity for local buckling.

2. Referenced Documents

The following documents or portions thereof are referenced within this Standard and shall be considered as part of the requirements of this document.

a. American Iron and Steel Institute (AISI), Washington, DC:

S100-07, *North American Specification for the Design of Cold-Formed Steel Structural Members*

S902-08, Stub-Column Test Method for Effective Area of Cold-Formed Steel Columns

b. ASTM International (ASTM), West Conshohocken, PA:

A370-07b, Standard Test Methods and Definitions for Mechanical Testing of Steel Products

E6-07b, Standard Terminology Relating to Methods of Mechanical Testing

IEEE/ASTM-SI-10-02, American National Standard for Use of the International System of Units (SI): The Modern Metric System

3. Terminology

Terms not defined in Section 3 of this Standard, AISI S100 or ASTM E6 shall have the ordinary accepted meaning for the context for which they are intended.

4. Symbols

F_y = Minimum specified design yield stress of column material
F_{yi} = Individual yield stresses used to compute F_{ya}
F_{ya} = Average yield stress of the sheet steel for a given test unit
L = Required test specimen length
P_u = Test column load at which failure occurs
P_{ua} = Average test column load

ϕ = Resistance factor
Ω = Safety factor

5. Apparatus

5.1 In lieu of a test machine, load shall be permitted to be applied by either a hydraulic or a pneumatic cylinder. When a cylinder is used, a calibrated load cell shall be used to measure the applied load to within ± 2 percent.

User Note:
The tests should be conducted on a testing machine that complies with the requirements of ASTM E4-07, Standard Practices for Force Verification of Testing Machines.

6. Test Unit

6.1 A test unit shall include a minimum of three nominally identical column specimens and a minimum of two corresponding sheet-type tensile specimens.

6.2 The specimens within a unit shall represent one type of cold-formed steel section with the nominally identical specified geometrical and mechanical properties. The specimens shall be permitted to be taken from the same column or from different production runs provided the source of the specimens is properly identified and recorded.

6.3 If column specimens are taken from different production runs, at least two corresponding sheet-type specimens shall be taken and tested from each production run.

6.4 The column test specimens shall be used to determine:

(1) The actual geometry of each specimen, and
(2) The nominal distortional buckling strength of a column.

6.5 The tensile test specimens shall be used to determine the yield stress of each column specimen according to the requirements described in ASTM A370.

6.6 For each test specimen and test unit, the measured geometrical and tested physical properties of the individual specimen shall meet the requirements stated by the fabricator and the material producer, respectively.

6.7 If the average area, thickness, or yield stress of a test unit varies by more than 20 percent from the respective nominal or specified-minimum value, the test unit shall be considered to be non-representative of the column section, and further evaluations are considered to be invalid.

7. Specimens

The column specimens shall meet length and end-flatness requirements as follows:

7.1 Column Length. The length requirements of the column test specimen, L, are that it be (1) short enough to minimize overall column buckling effects, and (2) long enough to minimize the end effects during loading. The required column length is defined by Section 7.1.1.

The length L is to be determined analytically or experimentally. If analytical determination of the test specimen length is used, the length is to be based on the minimum distortional buckling wave as determined by a finite strip or other appropriate finite element analysis. The specimen length with consideration of distortional buckling shall be at least four half wavelengths and shall be tested between flat ends. If the distortional buckling mode is not observed experimentally, the specimen length shall be adjusted to achieve the distortional buckling mode. If experimental determination of the test specimen length is used, the test specimen length shall be based on an array of tests of differing specimen lengths until the distortional buckling mode is observed or it is shown that distortional buckling is not a controlling limit state.

7.2 Column End Surface Preparation. The end planes of the column test specimens shall be carefully cut and milled to a flatness tolerance of plus or minus 0.002 in. (0.0508 mm).

7.3 Column Specimen Source. Column test specimens shall be cut from the commercially fabricated column product or shall be specially fabricated provided care is taken not to exceed the cold work of forming expected in the commercial product. If the specimen is specially fabricated, subsequent proof tests using specimens from commercially produced columns shall be required and reported.

7.4 Tensile Specimen Source. Longitudinal tensile specimens shall be cut from the center of the widest flat of a formed section from which the column specimens have been taken or from the sheet or coil material used for the fabrication of the column specimens. The tensile specimens shall not be taken from parts of a previously tested column.

8. Column Test Procedure

8.1 Care shall be taken to center the specimen on the axis of the test machine to ensure that the applied load is uniformly distributed over the specimen end surfaces. The column ends shall rest on flat steel plates, or on a spherical surface with a point contact, or on pins in mutually perpendicular directions, such that the resultant of the axial load is applied through the centroid of the gross section.

8.2 The load increments applied during the test shall not exceed 10 percent of the estimated maximum test load.

8.3 The maximum loading rate between load increments shall not exceed a corresponding applied stress rate of 3 ksi (21 MPa) of gross cross-sectional area per minute.

8.4 The test specimen shall be loaded to failure and the mode of failure shall be noted. Failure is the point at which the specimen will accept no additional load.

9. Calculations

9.1 For a given test unit, all individual test loads, P_u, derived from the column tests shall be used to calculate the average test load, P_{ua}. Similarly, all individual yield stresses, F_{yi}, derived from the tensile tests of the same unit shall be used to calculate the average yield stress of the same test unit, F_{ya}.

9.2 Extrapolations beyond 20 percent of the extreme parameters tested shall not be permitted.

10. Report

10.1 The report shall include a complete record of the sources and locations of all column and tensile-test specimens and shall describe whether the specimens were taken from one or several columns, one or several production runs, coil stock, or other sources.

10.2 The documentation shall include all measurements taken for each column test specimen, including (1) cross-section dimensions, (2) uncoated sheet thickness, (3) yield stress, (4) tensile strength, (5) percent elongation, (6) manufacturer, (7) end preparation, and (8) test and evaluation procedure used.

10.3 The determination of the selected column length shall be fully documented with appropriate calculations.

10.4 A description of the test setup and the instrumentation used to measure lateral displacements and axial shortening shall be included.

10.5 The report shall include the load increments, rate of loading, test loads and any observation made during the test for each column tested.

10.6 The report shall include complete calculations and results.

10.7 The report shall state any visual observations recorded that are pertinent to the performance of the test specimen(s).

10.8 The report shall describe any known deviations from this test method.

10.9 The report shall provide the data required (number of tests, coefficient of variation of test loads, etc.) to determine the resistance factor, ϕ, and safety factor, Ω, in accordance with Section F1 of the AISI S100.

11. Precision

The following criteria shall be used to judge the acceptability of the test results:

11.1 Repeatability. Individual column test results shall be excluded if they differ by more than 10 percent from the mean value for a test unit when tested with a minimum of three specimens.

11.2 Reproducibility. The results of tests on columns conducted at two or more laboratories are to agree within ten (10) percent when adjusted for differences in cross sectional dimensions and yield stress in order to be considered valid tests.

AISI S911-08

Method for Flexural Testing
Cold-Formed Steel Hat Shaped Beams

1. Scope

1.1 This method establishes test procedures for determining the nominal flexural strength of an open hat shaped cross section subject to negative bending moment.

1.2 This test method is permitted to be used to evaluate the nominal flexural strength of hat sections with or without a discrete intermediate bracing system.

1.3 This Standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of these standards to establish appropriate safety and health practices, and determine the applicability of regulatory limitations prior to use.

Commentary:
This test method can be used to establish the nominal flexural strength of a particular open hat shaped cross section which is subjected to local, distortional and/or overall buckling.

2. Referenced Documents

The following documents or portions thereof are referenced within this Standard and shall be considered as part of the requirements of this document.

a. American Iron and Steel Institute (AISI), Washington, DC:

S100-07, *North American Specification for the Design of Cold-Formed Steel Structural Members*

b. ASTM International (ASTM), West Conshohocken, PA:

A370-07b, Standard Test Methods and Definitions for Mechanical Testing of Steel Products

E6-07b, Standard Terminology Relating to Methods of Mechanical Testing

IEEE/ASTM-SI-10-02, American National Standard for Use of the International System of Units (SI): The Modern Metric System

3. Terminology

Where the following terms appear in this Standard they shall have the meaning as defined herein. Terms not defined in Section 3 of this Standard, AISI S100 or ASTM E6 shall have the ordinary accepted meaning for the context for which they are intended.

Negative bending moment. A moment which causes compression on the open side of the section.

Failure. A state at which the specimen will accept no additional load.

4. Symbols:

a = Measured distance along beam. See Figure 1
b = Measured distance along beam. See Figure 1
L = Span length of the section tested, measured center-to-center of end supports. See Figure 1
P_{ts} = Failure load of single span system tested

R = Support reaction
t = Nominal base steel thickness exclusive of coating
t_a = Average base steel thickness

5. Apparatus

5.1 The test method shall be generally suitable for either hydraulic or screw operated testing machines.

5.2 The test specimen support fixtures and the testing machine ram shall have the capability of maintaining a constant loading direction throughout the test.

5.3 The lateral support fixtures used in the test shall be installed in such a manner so as not to impede the horizontal displacement of the open side of the section, i.e. the compression flanges and the vertical deflection of the specimen.

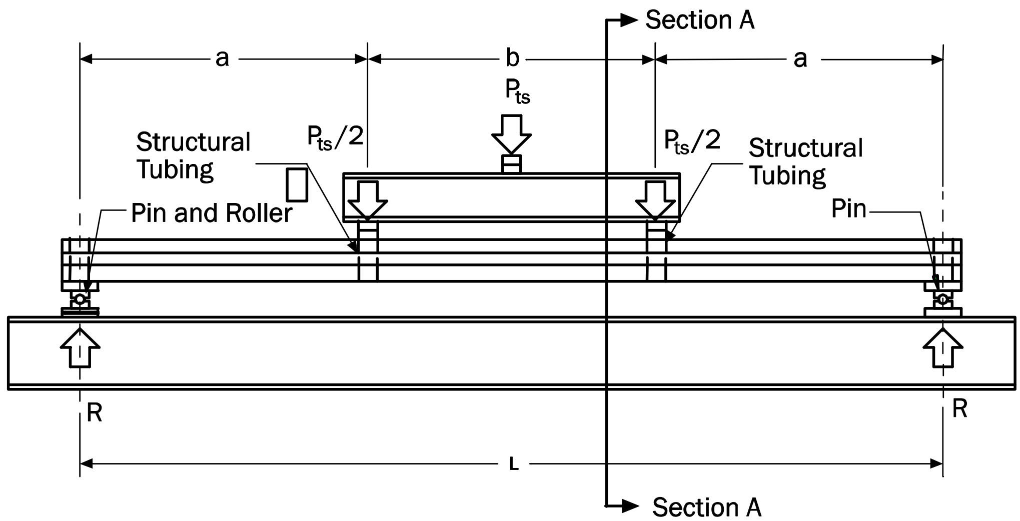

Figure 1 - Simply Supported Beam Test

5.4 In lieu of a test machine, the load shall be applied by either a hydraulic or a pneumatic cylinder. When a cylinder is used, a calibrated load cell shall be used to measure the applied load to within ± 2 percent.

User Note:
The testing machine should comply with the requirements of ASTM E4-07, Standard Practices for Force Verification of Testing Machines, wherein the rate of loading can be controlled, constant loads maintained, and the applied load can be measured accurately to within ± 2 percent.

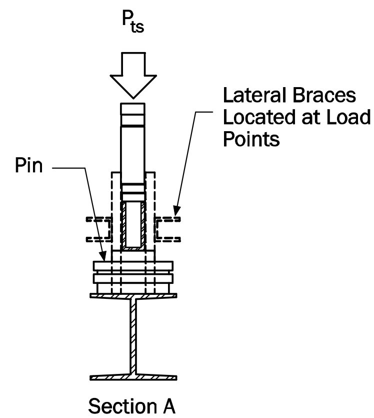

Figure 2 - Section of Simply Supported Beam Test

6. Test Unit

6.1 A test unit shall include a minimum of three identical beam specimens and a minimum of two corresponding sheet-type tensile specimens.

6.2 The specimens within a unit shall represent one type of cold-formed section with the same specified geometrical, physical, and chemical properties. The specimens shall be permitted to be taken from the same beam or from different production runs provided the source of the specimens is properly identified and recorded.

6.3 If beam specimens are taken from different production runs, at least two corresponding sheet-type specimens shall be taken and tested from each production run.

6.4 The test specimens shall be used to determine:

(1) The actual geometry of each specimen.

(2) The maximum beam test load.

6.5 The tensile test specimens shall be used to determine the yield stress, tensile strength, and percent elongation of each beam specimen in accordance with the requirements described in ASTM A370.

6.6 For each test specimen and test unit, the measured geometrical and tested mechanical properties of the individual specimens shall meet the requirements stated by the fabricator and material producer, respectively.

6.7 If the average area, thickness, or yield stress of a test unit varies by more than 20 percent from the specified-minimum value, the test unit shall be considered to be non-representative of the beam section, and further evaluations are considered to be invalid.

7. Specimens

7.1 The beam specimen shall be supported at one end by a pin condition and at the other end by a pin and roller condition. Lateral bracing at the beam specimen ends and at load points shall be permitted.

User Note:
It is permitted to restrain the top flange (open portion of a hat section in compression) at the load points to replicate the in-place, fabricated assembly.

7.2 The beam specimen shall be installed so as to cause compression on the open side of the hat section.

7.3 For tests including intermediate discrete point braces, the braces used in the test shall be installed in such a manner so as not to impede the lateral displacement of the compression flanges and the vertical deflection of the beam specimen.

7.4 Beam specimen length, "b", as shown in Figure 1, shall enable formation of each of the local buckling, distortional buckling, or overall buckling modes, and shall be determined as follows:

(1) For local buckling determination, length "b" is taken as at least three times the maximum flat width of the section.

(2) For overall buckling, length "b" is based on the maximum in-place unbraced length of the member.

(3) For distortional buckling, length "b" is determined analytically or experimentally. If the length is determined analytically, "b" is taken as a minimum of three half-wave lengths as determined analytically by a finite strip or finite element analysis, where the half-wave length is the one corresponding to the minimum distortional buckling. If the distortional buckling mode is not observed, the test specimen length is to be adjusted to achieve distortional buckling. If "b" is determined experimentally, an array of tests of differing lengths is performed until distortional buckling is observed. Length "a", as shown in Figure 1, is to be chosen to achieve the desired applied bending moment, but not less than three times the depth of the beam specimen.

7.5 At the point of application of the loads, the webs of the beam specimen shall be connected by self-drilling screws to a structural tube, or other element simulating the truss web in such a manner to effectively restrain lateral movement of the web. See Figure 1.

7.6 Beam Specimen Source. Beam test specimens shall be cut from the commercially fabricated beam product or beam test specimens shall be specially fabricated provided care is taken not to exceed the cold work of forming expected in the commercial product. If the beam test specimen is specially fabricated, subsequent proof tests using specimens from commercially produced beams shall be required and reported.

7.7 Tensile Specimen Source. Longitudinal tensile specimens shall be cut from the center of the widest flat of a formed section from which the beam specimens have been taken or the tensile specimens shall be taken from the sheet or coil material used for the fabrication of the beam specimens. The tensile specimens shall not be taken from parts of a previously tested beam.

8. Beam Test Procedure

8.1 The beam specimen shall be supported at one end by a pin condition and at the other end by a pin and roller condition. Lateral bracing at the beam specimen ends and at load points shall be permitted.

8.2 For tests including intermediate discrete point braces, the braces used in the test shall be installed in such a manner so as not to impede the lateral displacement of the compression flanges and the vertical deflection of the beam specimen.

8.3 A two-point load shall be applied to the system to produce a negative bending moment in the test specimen. See Figure 1.

8.4 Care shall also be taken to center the specimen on the axis of the test machine.

8.5 The load increments applied during the test shall not exceed 10 percent of the estimated maximum test load.

8.6 The test specimen shall be loaded to failure, P_{ts}, and the mode of failure reported.

8.7 The maximum loading rate between load increments shall not exceed a corresponding applied stress of 3 ksi (21 MPa) at the extreme fiber of the gross cross section per minute.

8.8 Deflections of the specimen shall be permitted to be measured during the test. When deflections are recorded, the following procedures shall be required:

(1) The deflection shall be measured to the nearest 0.001 in (0.0254 mm) at each load increment, and

(2) The load increments applied during the test shall be the same for each specimen within a test unit, with a variation not to exceed one percent.

9. Calculations

9.1 Extrapolations beyond 20 percent of the extreme parameters tested shall not be permitted.

10. Report

10.1 The report shall include a complete record of the sources and locations of all beams and tensile-test specimens, and shall describe whether the specimens were taken from one or several beams, or several production runs, coil stock, or other sources.

10.2 The documentation shall include all measurements taken for each beam test specimen, including (1) cross-section dimensions, (2) uncoated sheet thickness, (3) yield stress, (4) tensile strength, (5) percent elongation, (6) applicable material specification, (7) manufacturer, (8) test setup characteristics such as lateral brace locations and bearing stiffeners, and (9) and evaluation procedure used.

10.3 The determination of the selected beam span shall be fully documented with appropriate calculations.

10.4 A description of the test setup and the instrumentation used shall be included.

10.5 The report shall include the load increments, rate of loading, ultimate loads and observation made during the test for each beam tested.

10.6 The report shall include complete calculations and results.

10.7 The report shall state any visual observations recorded that are pertinent to the performance of the test specimen(s).

10.8 The report shall provide the data required (number of tests, coefficient of variation of the test load, etc.) for the determination of resistance factor, ϕ, and safety factor, Ω, in accordance with Section F1 of AISI S100.

11. Precision

The following criteria shall be used to judge the acceptability of the test results:

11.1 Repeatability. The individual beam test results shall be considered suspect if they differ by more than ten (10) percent from the mean value for a test unit with at least three specimens.

11.2 Reproducibility. If tests are performed at different test labs, the results of tests on beams conducted at two or more laboratories shall agree within ten (10) percent when adjusted for differences in cross sectional dimensions and yield stress in order to be considered valid tests.

AISI S912-08

Test Procedure for Determining a Strength Value for a Roof Panel-to-Purlin-to-Anchorage Device Connection

1. Scope

1.1 The purpose of this test is to obtain lower bound strength values for the roof panel-to-purlin-to-anchorage device connections in through-fastened and standing seam, multi-span, multi-purlin line roof systems, with or without intermediate braces. The test is not intended to determine the ultimate strength of the connections.

1.2 This test method applies to an assembly consisting of through-fastened or standing seam panels, purlins of C or Z-sections, and anchorage devices.

1.3 The test procedure is only for gravity loading cases and only for a series of purlins with flanges facing in the same direction. It applies only to the anchorage configurations described in Section D6.3.1 of AISI S100.

1.4 All tests are conducted using roof panels, clips, fasteners, insulation, thermal blocks, discrete braces, and purlins as used in the actual roof system except as noted in Section 1.5.

1.5 Tests conducted with insulation are applicable to identical systems with thinner or no insulation.

Commentary:
Due to the many different types and methods of construction of steel roof systems, it is not practical to develop a generic method to predict the strength of the roof panel-to-purlin-to-anchorage device connections. The interaction of the three components near an anchorage location is a complex phenomenon and highly indeterminate.

The test method provides designer with a means of establishing a lower bound on the strength of the roof panel-to-purlin-to-anchorage device connections. An appropriate strength reduction factor or safety factor should be applied to test results for design use.

2. Referenced Documents

The following documents or portions thereof are referenced within this Standard and shall be considered as part of the requirements of this document.

a. American Iron and Steel Institute (AISI), Washington, DC:

S100-07, *North American Specification for the Design of Cold-Formed Steel Structural Members*

S908-08, Test Standard for Base Test Method for a Standing Seam Roof System

b. ASTM International (ASTM), West Conshohocken, PA:

A370-07b, Standard Test Methods and Definitions for Mechanical Testing of Steel Products

E6-07b, Standard Terminology Relating to Methods of Mechanical Testing

IEEE/ASTM-SI-10-02, American National Standard for Use of the International System of Units (SI): The Modern Metric System

3. Terminology

Where the following terms appear in this Standard they shall have the meaning as defined herein. Terms not defined in Section 3 of this Standard, AISI S100 or ASTM E6 shall have the ordinary accepted meaning for the context for which they are intended.

Anchorage system. A series of components which carry forces in the roof sheathing to the primary structural system of the building.

Failure. A state at which the specimen will accept no further loading.

Field erection drawings. Drawings issued by a metal roof manufacturer to field erectors showing parts and assembly procedure.

Fixed clip. A hold down clip which does not allow roof panel to move independently of the roof substructure.

Insulation. Glass fiber blanket or rigid board.

Lateral. A direction normal to the span of the purlins in the plane of the roof sheathing.

Thermal block. Strips of rigid insulation located directly over the purlin between clips.

Pan type standing seam roof. A "U" shaped panel which has vertical sides.

Rib type standing seam roof. A panel which has ribs with sloping sides and forms a trapezoidal shaped void at the side lap.

Sliding clip. A hold down clip which allows the roof panel to move independently of the roof substructure.

Standing seam roof system. A roof system in which the side laps between the roof panels are arranged in a vertical position above the roof line. The roof panel system is secured to the purlins by means of concealed hold down clips that are attached to the purlins with mechanical fasteners.

Test engineer. Engineer or designated representative responsible for supervising the test assembly, collection of test data, and preparation of test report.

Through fastened roof system. A roof system in which the side laps between the roof panels are arranged in a vertical position above the roof line. The roof panel system is secured to the purlins by means of self-drilling or self-tapping fasteners through the panels and into the purlins.

4. Units of Symbols and Terms

Any compatible system of measurement units shall be permitted to be used in this Standard, except where explicitly stated otherwise. The unit systems considered in this Standard shall include U.S. customary units (force in kips and length in inches) and SI units (force in Newtons and length in millimeters) in accordance with IEEE/ASTM-SI-10.

5. Apparatus

5.1 A test setup shall be capable of supporting simulated gravity loading. Loading shall be applied by differential pressure or by using weight. The test assembly shall be permitted to be flat or sloped as required to determine the strength value in the up slope or down slope direction.

5.2 The length, width, number of purlins, and number of spans of the test setup shall be at the discretion of the test engineer. The height of the assembly shall be enough to permit assembly of the specimen and to ensure adequate clearance at the maximum deflection of the specimen.

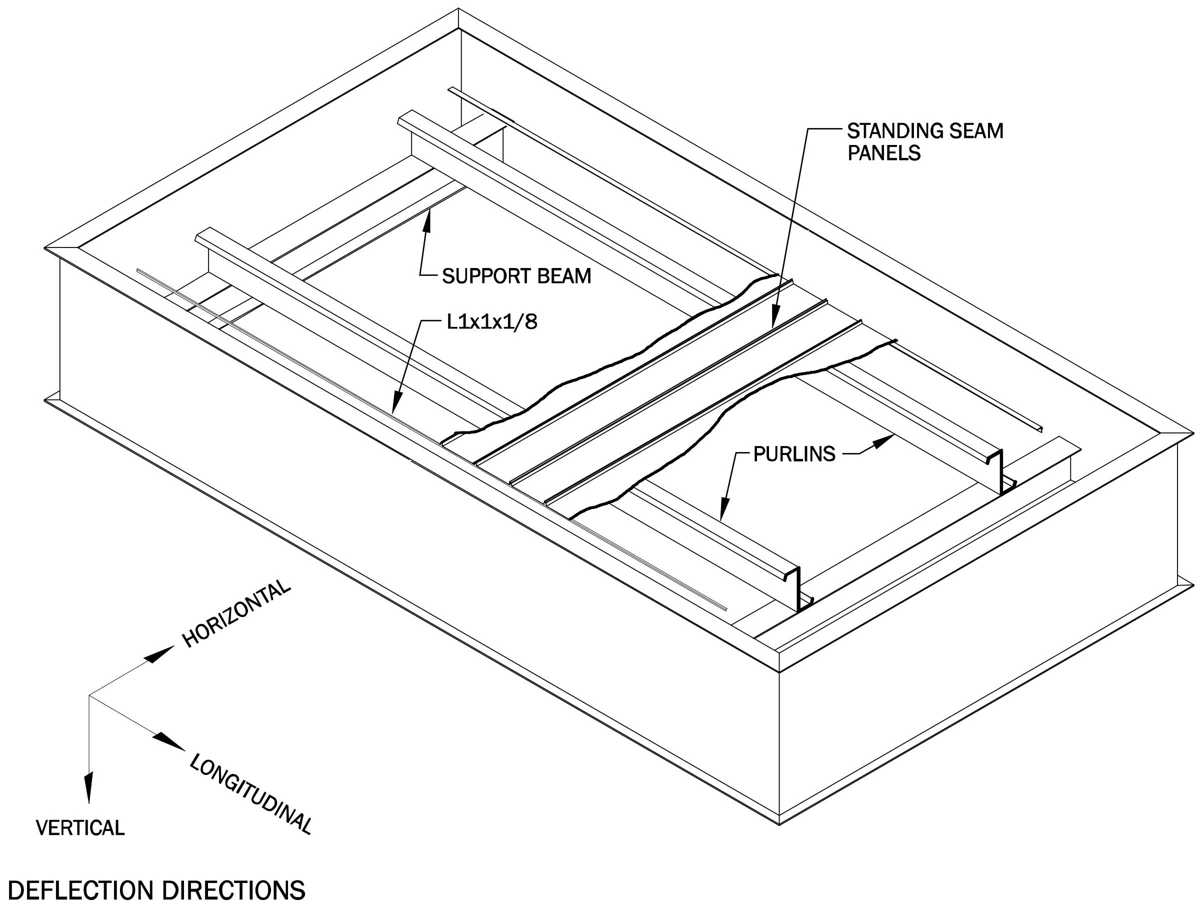

Figure 1 – Test Chamber

5.2.1 Differential Pressure Loading Procedure

A rectangular vacuum box shall be constructed of any material with enough strength and rigidity to provide the desired pressure differential without collapse. See Figure 1 for a typical test chamber. Other chamber orientations shall be permitted.

The width of the chamber shall be determined by the maximum panel length. Allowance shall be made in the interior chamber dimensions to accommodate structural supports for the secondary members and enough clearance on all sides to prevent interference of the chamber wall with the test specimen as it deflects.

Sections 6.4 to 6.9 of AISI S908 shall be followed.

5.2.2 Weight Loading Procedure

Uniformly distributed weights placed in increments shall be used. Placement of the weights shall be such that bridging does not occur. The weights shall be permitted to be small steel plates, concrete or masonry bricks, or larger plates or rods whose bending stiffness is less than 20% of the bending stiffness of the sheathing.

Sections 6.4 to 6.9 of AISI S908 shall be followed.

6. Test Specimens

6.1 Test purlins shall be supported at each end by a steel beam. The beams shall be simply supported and all but one of the beams shall be sufficiently free to translate laterally to relieve any longitudinal catenary forces in the assembly. Purlins shall be connected to the supporting beams as shown in the field erection drawings.

6.2 Panel supporting clips, fasteners, and panels shall be installed as required by the field erection drawings.

6.3 Means of providing purlin restraint shall be as required for use in actual field application, and shall be installed as shown on the field erection drawings.

6.4 For tests including intermediate discrete point braces, the braces used in the test shall be installed in such a manner so as not to impede the vertical deflection of the specimen.

6.5 For standing seam roof systems, a 1 in. x 1 in. (25 mm x 25 mm) continuous angle with a maximum thickness of 1/8 in. (3 mm) or a member of compatible stiffness shall be attached to the underside at each end of the panels to prevent separation of the panels at the ends of the seam. Fasteners shall be placed on both sides of each major rib.

6.7 Panel joints shall not be taped and no tape shall be used to restrict panel movement.

7. Test Procedure

7.1 A test series shall be conducted for each roof panel-to-purlin-to-anchorage device system. The setup shall consist of any number of purlin lines and any number of purlin spans. All purlin flanges shall face in the same direction. The anchorage system shall be located along an external purlin line and shall be permitted to consist of any of the anchorage combinations specified in Section D6.3.1 of AISI S100.

7.2 A test series shall consist of no fewer than three tests for each anchorage system.

7.3 The physical properties of all components shall be measured and recorded prior to testing. The yield stress of the panel, purlin and anchorage device material used in the tests shall be determined in accordance with ASTM A370. Coupons shall not be taken from areas where cold-working stresses could affect the results.

7.4 To simulate gravity loading, differential pressure or weights shall be applied to the system to produce simulated gravity loading moments in the system.

7.5 An initial load equal to 5 psf (0.25 kPa) shall be applied and removed to set the zero readings before actual system loading begins.

7.6 It shall not be required to load the system to failure. If it is loaded to failure, the mode of failure shall be noted. If the test must be stopped due to a flexural failure of the panel or purlin, or web crippling of the purlin, the result shall be permitted to be included in the test program.

7.7 Horizontal deflection near the top of each anchorage device shall be measured. Vertical deflection measurements shall be taken at the mid-span of at least two purlins in each span. The deck deflection in the horizontal direction shall be measured at the seam joint nearest the center of each span of the test assembly.

7.8 Deflections and loads shall be recorded at loading intervals equal to a maximum of 10 percent of the anticipated maximum load.

8. Test Evaluation

8.1 The lower bound strength of each roof panel-to-purlin-anchorage device connection used in the test shall be determined by calculating the anchorage force, P_L, at that location using the provisions in Section D6.3.1 of AISI S100. The lesser of load corresponding to a measured deflection of ½ in. (13 mm) at the top of the anchorage device or the maximum applied load in the test shall be used for this calculation.

8.2 The nominal strength of the panel-to-purlin-to-anchorage device connections shall be taken as the mean of the calculated anchorage forces minus one standard deviation.

8.3 The lower bound available strength shall be determined using a resistance factor, ϕ, of 0.9 or factor of safety, Ω, of 1.67.

9. Test Report

9.1 The report shall include who performed the test and a brief description of the system being tested.

9.2 The documentation shall include all test details with a drawing showing the test assembly and indicating the components and their locations, and the locations of all instrumentation. A written description of the test setup detailing the basic concept, loadings, measurements, and assembly shall be included.

9.3 The report shall include a drawing showing the measured geometry of all components and nominal material specifications. Material test results defining the actual material properties - material thickness, yield stress, tensile strength, and percent elongation shall be included.

9.4 The report shall include the test designation, loading increments, all measured deflections, maximum applied load or failure load and failure mode if failure occurred, and a description of the condition of each assembly at the end each test.

9.5 The report shall include calculations used to determine the lower bound strength for each test and the nominal strength of the roof panel-to-purlin-to-anchorage device connection tested, a description summarizing the test program results to include specimen type, span, and the supporting calculations.

AISI S913-08

Test Standard for Hold-Downs Attached to Cold-Formed Steel Structural Framing

1. Scope

1.1 This Standard provides methodology to determine both the strength and deformation behavior of hold-downs used in cold-formed steel light-frame construction.

User Note:

1. This Standard is specifically applicable to hold-down devices as employed in lateral load resisting shear walls. If the Standard is used for other applications, the engineer of record or the test agency must define the applicable limits as required in Section 10.1.2 and 10.1.3.
2. Illustrated in Figure 1 are some typical hold-downs, but there are many other configurations.

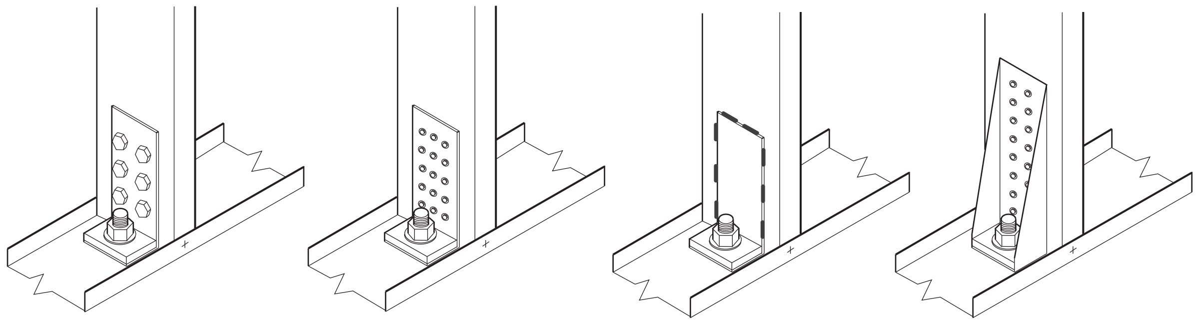

Figure 1 - Typical Hold-down Assemblies

1.2 Strength of hold-downs is determined by testing a hold-down device using a steel fixture (Section 8.2) or by testing a hold-down assembly in accordance with this Standard.

1.3 Deformation of hold-downs is determined by testing a hold-down assembly (Section 8.3) in accordance with this Standard.

1.4 This Standard applies to hold-downs attached to the cold-formed steel structural framing by use of welds or fasteners and to the supporting structure using anchor bolt/rod(s).

1.5 This Standard consists of Sections 1 through 11 inclusive.

2. Referenced Documents

The following documents or portions thereof are referenced within this Standard and shall be considered as part of the requirements of this document.

a. American Iron and Steel Institute (AISI), Washington, DC:

AISI S100-07, *North American Specification for the Design of Cold-Formed Steel Structural Members*

b. ASTM International (ASTM), West Conshohocken, PA:

A370-07b, Standard Test Methods and Definitions for Mechanical Testing of Steel Products

E6-07b, Standard Terminology Relating to Methods of Mechanical Testing

IEEE/ASTM-SI-10-02, American National Standard for Use of the International System of Units (SI): The Modern Metric System

3. Terminology

Where the following terms appear in this Standard, they shall have the meaning as defined herein. Terms not defined in Section 3 of this Standard, AISI S100 or ASTM E6 shall have the ordinary accepted meaning for the context for which they are intended.

Connection. Combination of structural elements and joints used to transmit forces between two or more members.

Fastener. Bolts, screws, power-driven pins or nails, clinches, or other mechanical fasteners.

Hold-down. Device used to resist uplift of the chords of shear walls, uplift on cold-formed steel members resisting uplift or lateral loads for wall anchorage.

Hold-down Device. See hold-down.

Hold-down Assembly. Assembly consisting of the following components: (1) a hold-down device, (2) an anchor bolt/rod(s) attached to the seat of the device, (3) cold-formed steel member(s) having specified dimensions and properties, (4) fasteners or welds used to attach the hold-down device to the cold-formed steel member(s), and, if applicable, (5) bearing plates or washers used to enhance the performance of the hold-down assembly.

4. Units of Symbols and Terms

Any compatible system of measurement units shall be permitted to be used in this Standard, except where explicitly stated otherwise. The unit systems considered in this Standard shall include U.S. customary units (force in kips and length in inches) and SI units (force in Newtons and length in millimeters).

5. Precision

5.1 Loads shall be recorded to a precision of 1 percent of the anticipated ultimate load during application of test loads.

5.2 Deflections shall be recorded to a precision of 0.001 in. (0.025 mm).

6. Test Fixture

The test fixture shall consist of either:

(a) A hydraulic or screw operated testing machine capable of operating at a constant rate of motion of the movable crosshead or a constant rate of loading, and a calibrated force-measuring device, or

(b) A hydraulic cylinder with a steel fixture, and a calibrated load cell.

User Note:
It is recommended that ASTM E4-07, Standard Practices for Force Verification of Testing Machines, be used as applicable.

7. Test Specimen

7.1 The test specimen shall consist of the tested hold-down device and, in the case of the hold-down assembly test, the fasteners used to connect the hold-down device to the cold-formed steel member(s) specified for use with the hold-down device.

7.2 The number of specimens tested shall comply with the requirements of Section F1 of AISI S100.

7.3 The steel properties of the tested hold-down device, including yield stress, tensile strength, percent elongation, and uncoated base steel thickness shall be determined. Standard tensile tests of the steel from which the hold-down device was produced shall be conducted in accordance with ASTM A370 and Section F1.1(c) of AISI S100.

7.4 Fasteners used in hold-down assembly testing shall be selected from one manufacturer's lot at random and installed in a manner that is representative of field conditions.

7.5 Welding, clinching or other fastening techniques shall be permitted in a manner that is representative of field conditions.

8. Test Setup

8.1 General

8.1.1 Hold-down devices and assemblies shall be tested individually in such a manner to simulate the essential function of the hold-down device or assembly. Test loads shall be applied with reference to the intended end-use application of the hold-down device or assembly.

8.1.2 The anchor bolt/rod, which is attached to the seat of the hold-down device, shall be fastened to the test apparatus in such a manner that the connection to the test bed does not affect the test results. Additionally, the anchor bolt/rod shall be installed through the hole in the bearing seat of the hold-down device and attached to the device with a nut and washer in accordance with the end-use application as prescribed by the manufacturer's installation instructions.

8.1.3 When testing a bolted hold-down device or assembly, the nuts used with the bolts to the steel fixture or cold-formed steel member(s) shall only be snug tight, to remove the effects of clamping, unless a torque is specified in the end use.

8.2 Hold-down Device Test Using a Steel Fixture (Figure 2)

8.2.1 For tension load testing, the anchor bolt/rod shall have a minimum length of 1.0 in. (25.4 mm), measured from the seat of the hold-down device to the test bed (see Figure 2(a)). Alternatively, the hold-down device shall be permitted to be installed directly (e.g., flush) on the test bed if this is consistent with manufacturer's installation instructions (See Figure 2(b)).

8.2.3 To minimize friction forces between the hold-down device and the steel fixture, a low friction material such as Teflon® or polyethylene shall be inserted between the device and steel fixture before load testing.

8.2.4 Bolt(s), used to attach the device to the steel fixture, shall be permitted to be higher strength than specified for intended use with no load reduction to preclude failure prior to hold-down device failure; however, the bolt(s) diameter, nut, and washer, if used, dimensions shall be in accordance with the end-use application and the manufacturer's installation instructions.

8.2.5 Anchor bolt/rod(s) shall be permitted to be higher strength than specified for intended use with no load reduction to preclude failure prior to hold-down device failure; however, the anchor bolt/rod(s) diameter and the nut dimensions shall be in accordance with the end-use application (manufacturer's installation instructions). For optional compression testing, the nut and washer dimensions shall be in accordance with the end-use application and the manufacturer's installation instructions.

8.2.6 For optional compression load testing with the hold-down device in the raised setting condition (see Figure 2(a)), the maximum unbraced length of the test anchor bolt/rod(s) intended for use with hold-down device shall be as specified by the hold-down manufacturer. For the hold-down device in the raised setting condition (Figure 3(a)), a minimum of a 1.0 in. (25.4 mm) gap shall be provided between the hold-down device and the test bed.

User Note:
The unbraced length of the anchor bolt/rod may be taken as the distance between the nut below the hold-down and the nut above the test bed that is installed for the compression test.

8.2.7 When testing a hold-down device on a steel fixture, the deflection measurement device shall measure the relative movement between the hold-down device and the steel test fixture. Displacement shall be measured between the top of the anchor bolt/rod attached at the seat of the device and a fixed reference point on the steel fixture just above the device (See Figure 2).

8.2.8 An additional force measurement device shall be used to measure the anchor bolt force when the hold-down device is installed directly (i.e., flush) on the test bed.

Commentary:
Hold-down devices installed directly (i.e., flush) on a rigid base such as the test bed or a concrete foundation are subject to prying action. Depending on the configuration of the hold-down, the anchor bolt force may be indeterminate.

8.2.9 Low friction material shall be permitted to be placed at the top and bottom of the hold-down device or steel fixture to resist horizontal forces that may be a result of eccentricities in the test setup.

8.3 Hold-down Assembly Test (Figure 3)

8.3.1 The hold-down assembly test setup shall consist of the cold-formed steel member(s) specified for use with the hold-down device; welds or fasteners specified for attaching the device to the cold-formed steel member(s); and, for tension load testing, an anchor bolt/rod having a minimum length of 1.0 inch (25.4 mm), measured from the seat of the hold-down device to the test bed (see Figure 3(a)). Alternatively, the hold-down shall be permitted to be installed directly (i.e., flush) on the test bed if this is consistent with manufacturer's installation instructions (see Figure 3(b)).

8.3.2 Installation of the hold-down device to the cold-formed steel member(s) shall maintain fastener end and edge distances as expected in field conditions.

8.3.3 Specified bolt strength used in the test shall be in accordance with the end-use application and the manufacturer's installation instructions.

8.3.4 Anchor bolt/rod(s) shall be permitted to be higher strength than specified for intended use with no load reduction to preclude failure prior to hold-down device or cold-formed steel member(s) failure; however, the anchor bolt/rod(s) diameter and the nut dimensions shall be in accordance with the end-use application and the manufacturer's installation instructions.

8.3.5 For optional compression load testing, the maximum unbraced length of the test anchor bolt/rod, intended for use with hold-down assembly shall be as specified by the hold-down manufacturer. The nut and washer dimensions shall be in accordance with the end-use application and the manufacturer's installation instructions. For the hold-down assembly in the raised setting condition (Figure 3(a)), a minimum of a 1.0 in. (25.4 mm) gap shall be provided between the hold-down device and the test bed.

User Note:
The unbraced length of the anchor bolt/rod may be taken as the distance between the nut below the hold-down and the nut above the test bed that is installed for the compression test.

8.3.6 For optional compression load testing with the hold-down assembly in the raised or flush setting conditions (Figure 3(a) and 3(b)), a minimum of a 1.0 in. (25.4 mm) gap shall be provided between the cold-formed steel member(s) and test bed to ensure that the bearing strength of the cold-formed steel member(s) is not included in the available compression strength of the hold-down.

8.3.7 When testing a hold-down assembly to a cold-formed steel member(s), the deflection measurement device shall measure the relative movement between the cold-formed steel member(s) and the test bed. Displacement shall be measured between the cold-formed steel member(s) and a fixed reference point on the test bed where the anchor bolt/rod is attached. Placement of the deflection measurement device shall ensure accurate measurement of the assembly displacement that includes deformation and rotation of the body of the hold-down device, slip between the device and the cold-formed steel member(s), and fastener slip (and anchor bolt/rod elongation, when applicable). See Figure 3.

8.3.8 A force measurement device shall be used to measure the anchor bolt force when the hold-down device is installed directly (i.e., flush) on the test bed.

Commentary:
Hold-down devices installed directly on a rigid base, such as the test bed or a concrete foundation, are subject to prying action. Depending on the configuration of the hold-down, the anchor bolt force may be indeterminate.

8.3.9 Low friction material shall be permitted to be placed at the top and bottom of the hold-down device or cold-formed steel member(s) to resist horizontal forces that may be a result of eccentricities in the test setup.

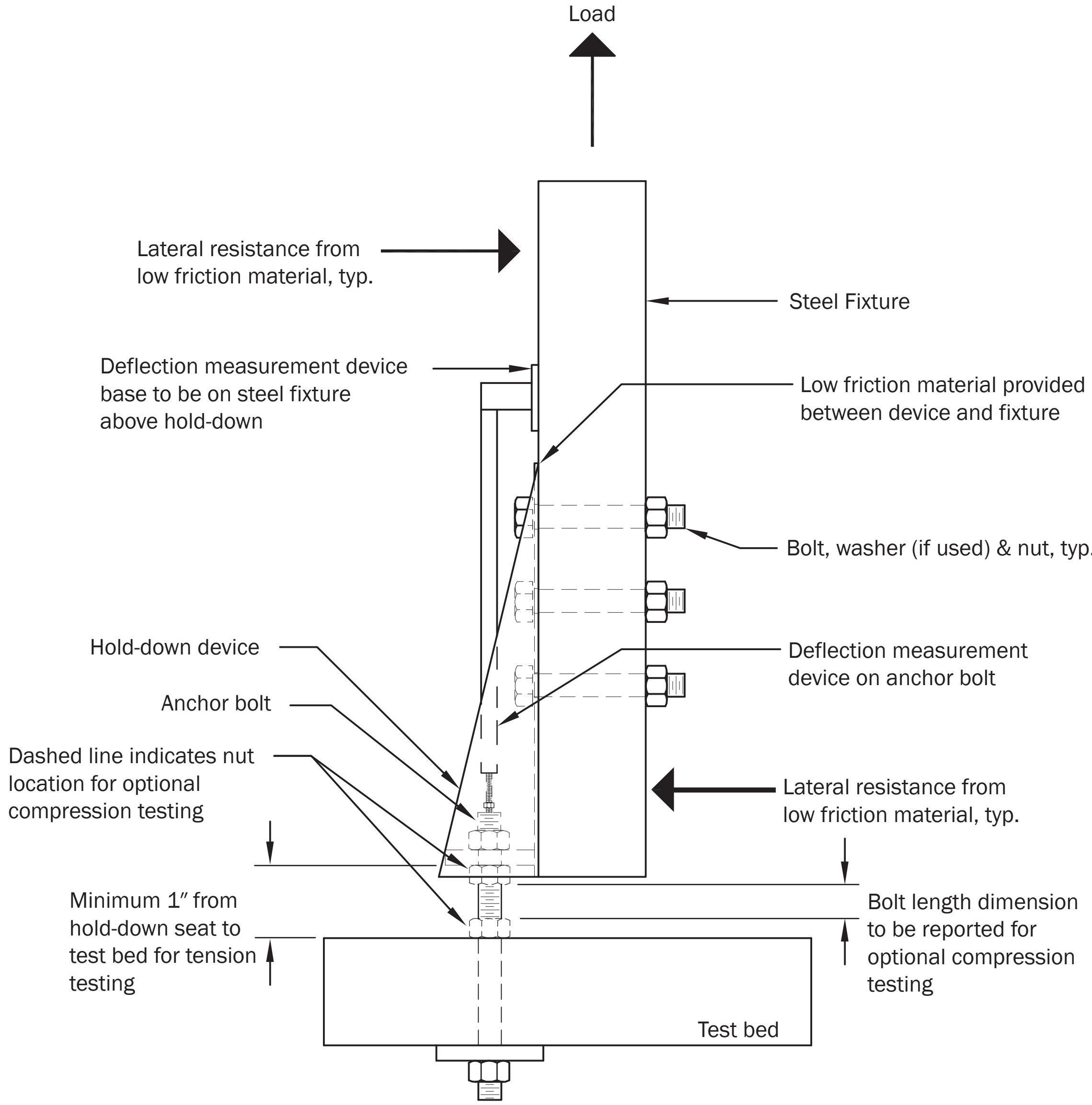

Figure 2(a) - Tension Load Test Set-up for a Single Hold-down Device

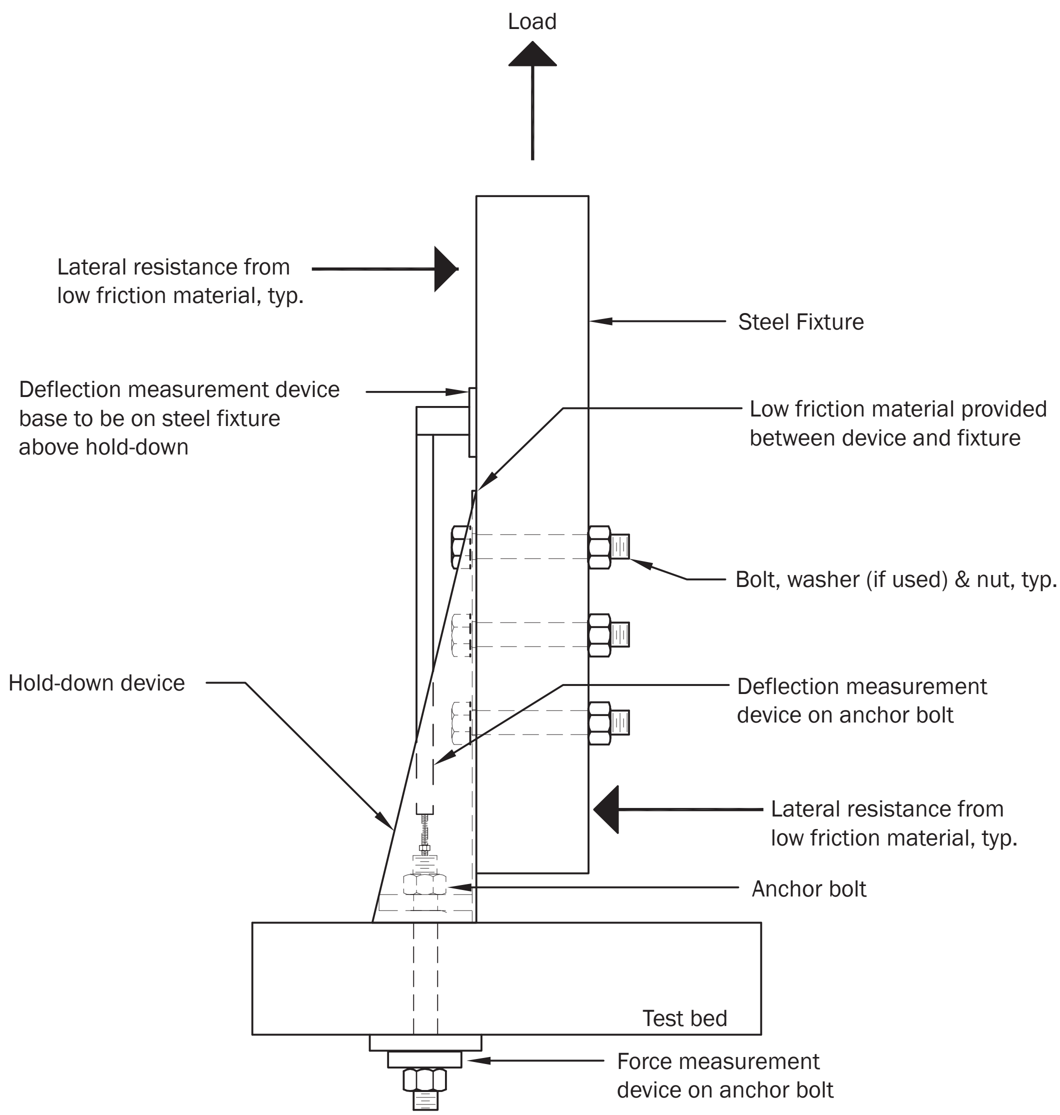

Figure 2(b) - Tension Load Test Set-up for a Single Hold-Down Device Flush to Test Bed

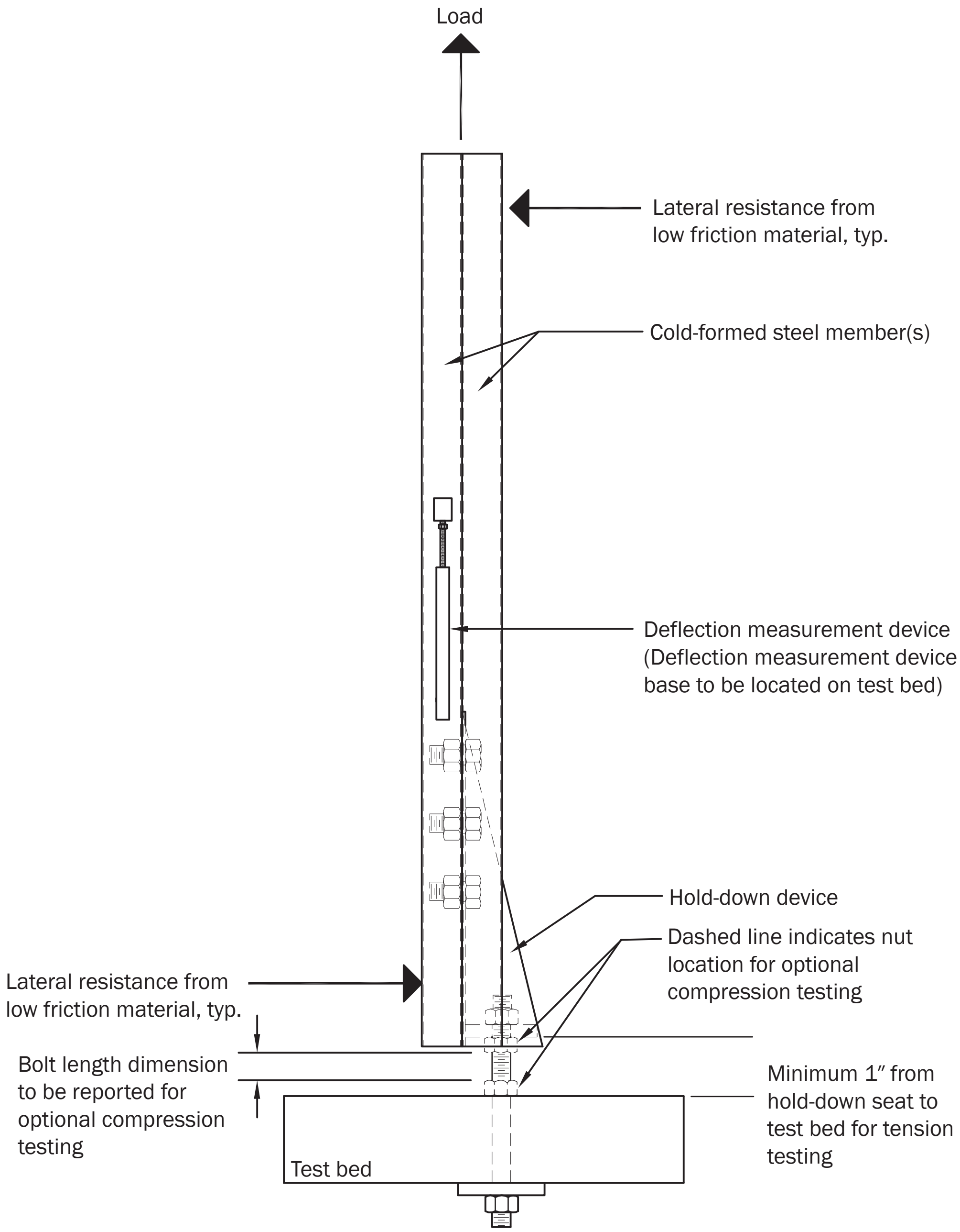

Figure 3(a) - Tension Load Test Set-up for a Single Hold-down Assembly

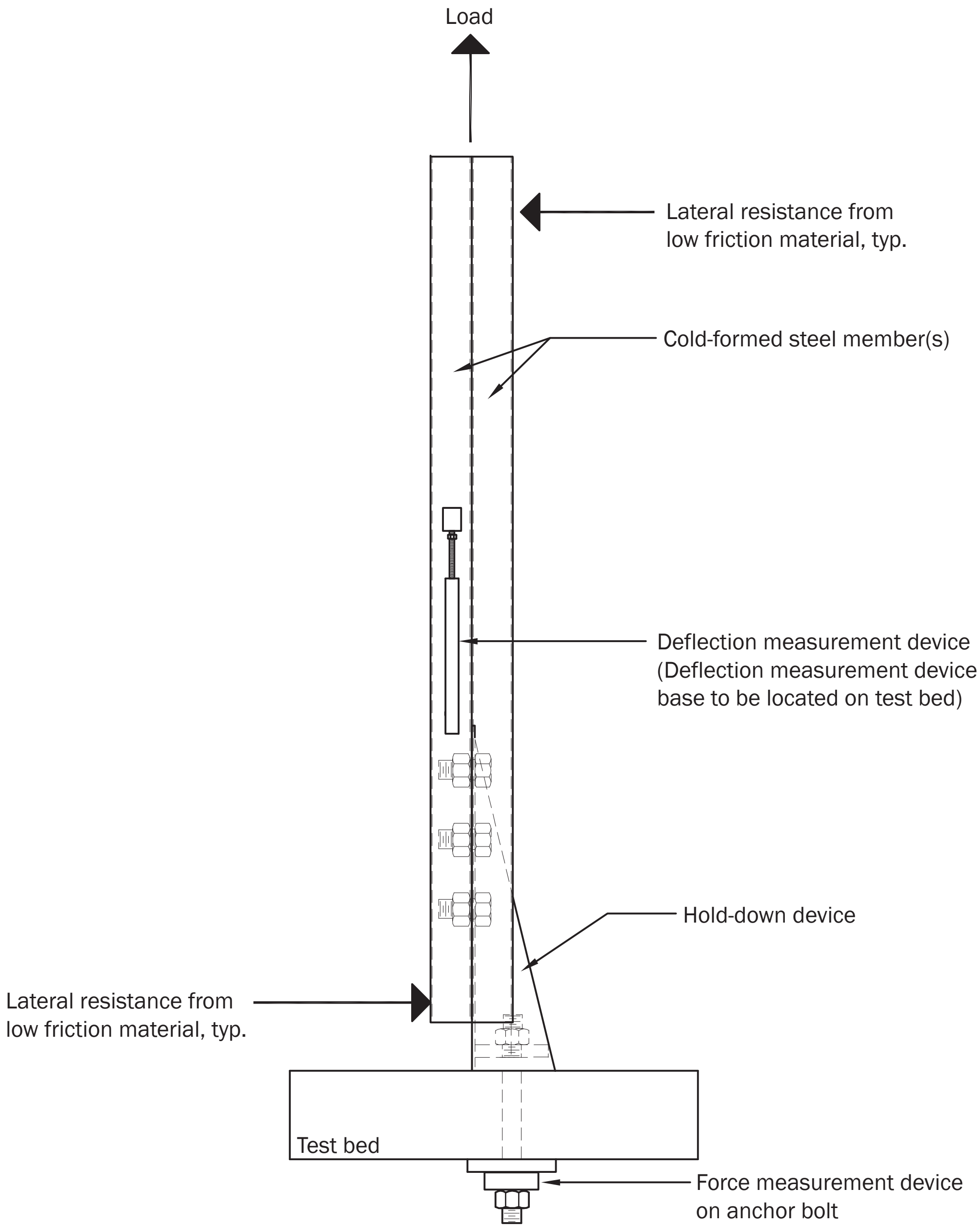

Figure 3(b) - Tension Load Test Set-up for a Single Hold-Down Assembly Flush to Test Bed

9. Test Procedure

9.1 An initial load, or preload, shall not be applied for tension (uplift) load or compression load (optional) testing of hold-down devices or hold-down assemblies.

Commentary:
An initial load or pre-load is not representative of field conditions.

9.2 The test load shall be applied at a uniform rate between 0.03 in. and 0.20 in. (0.8 mm and 5.1 mm) per minute until failure or maximum load. Loads shall be recorded to a precision of ± 1 percent of the ultimate load during application of test loads.

9.3 The displacements shall be recorded to the nearest 0.001 in. (0.025 mm). Deflections shall be recorded at a sufficient number of load levels to permit the establishment of a load-deflection curve. At least eight readings shall be taken prior to reaching the deflection limit state and the tested deflection limit shall be recorded. Readings shall be taken throughout the test and not be grouped such as at the beginning, middle or end of the test.

9.4 Tension Load Test

9.4.1 Hold-down devices shall be tested such that a tension load is applied in reference to the intended application of the device where attached to a steel fixture as described in Section 8.2.

9.4.2 Hold-down assemblies shall be tested such that a tension load is applied in reference to the intended application of the assembly where attached to a cold-formed steel member(s) as described in Section 8.3.

9.5 Compression Load Test

9.5.1 Compression load testing shall be conducted where the intended use of the hold-down device will include resistance to compression.

9.5.2 Hold-down devices shall be tested such that a compression load is applied in reference to the intended application of the device where attached to a steel fixture as described in Section 8.2.

9.5.3 Hold-down assemblies shall be tested such that a compression load is applied in reference to the intended application of the assembly where attached to a cold-formed steel member(s) as described in Section 8.3.

10. Data Evaluation

10.1 The available strength (i.e., allowable strength and/or design strength [resistance]*) based on tests shall be the least value determined in accordance with Sections 10.1.1, 10.1.2 and 10.1.3, as applicable.

10.1.1 The available strength shall be determined in accordance with the procedures described in Section F1 of AISI S100.

* Bracketed terms are equivalent terms that apply particularly to Limit States Design (LSD). See AISI S100 for further information.

User Note:
Table F1 of Section F1 of AISI 100 provides statistical data. When the device itself fails, then the statistical values for "Structural members not listed above" must be used to evaluate the allowable strength and/or design strength.

10.1.2 For hold-downs used in shear walls or those that otherwise contribute to the story drift, allowable strength shall be determined by averaging the test load at the applicable deflection limit prescribed in Section 10.5 and multiplying by 0.7.

Commentary:
Hold-down devices are load rated based on the lesser of a strength limit state and a deflection limit state. The allowable strength adjustment factor of 0.7 is taken from the load combinations of ASCE 7-05, *Minimum Loads for Buildings and Other Structures*, to adjust the deflection limit state value for Allowable Strength Design (ASD). The 0.7 is the inverse of the earthquake load factor 1.4 used in Alternative Basic Load Combinations.

10.1.3 For hold-downs used in shear walls or those that otherwise contribute to the story drift, design strength [resistance] shall be determined by averaging the test load at the applicable deflection limit prescribed in Section 10.5.

10.2 No test result shall be eliminated unless a rationale for its exclusion can be given.

10.3 The hold-down device test using a steel fixture or the hold-down assembly test shall be used to determine the tested strength of the hold-down. The strength of the hold-down connection shall be the lowest of the tested strength, the strength of the cold-formed steel member that the hold-down is to be attached to, or the strength of the screw, bolt, and/or welded connections as determined from the applicable section of the AISI S100. The hold-down assembly test shall be required to determine the strength of the hold-down connection where the hold-down connection to the cold-formed steel members is with fasteners other than what is recognized in the AISI S100. The hold-down assembly test shall be permitted to be used to determine the tested strength of the hold-down in lieu of the hold-down device test.

10.4 The hold-down assembly test, in which the hold-down is attached to cold-formed steel members, shall be used to obtain the displacement of the hold-down connection inclusive of the hold-down displacement.

10.5 The deflection limit for hold-downs used in shear walls or those that otherwise contribute to story drift shall be 0.185 in. (4.7 mm) for the hold-down device test and 0.25 in. (6.4 mm) for the hold-down assembly test, unless otherwise defined by the applicable building code or a design standard approved by the authority having jurisdiction.

Commentary:
These deflection limits are based on traditionally accepted values for the seismic design of shear walls, noting that the seismic story drift is to be checked at the strength level in accordance with ASCE 7. An 1/8 in. (3.18 mm) deflection limit for ASD was generally used for hold-downs attached to a steel test apparatus and this was increased to 3/16 in. (4.76 mm) to account for fastener slip when the hold-down was tested to actual studs. As it is desired to have a LRFD as well as an ASD load rating, the 1/8 in. (3.18 mm) and 3/16 in. (4.76 mm) deflection limits were increased to 0.185 in. (4.70 mm) and 0.25 in. (6.35 mm). The load at these higher deflection limits is the LRFD deflection load limit and one multiplies the load at these deflection values by 0.7 to determine the ASD deflection load limit (see Section 10.1.2). This test standard conservatively applies these limits to all load applications in order to avoid multiple load ratings for the same hold-down.

11. Test Report

11.1 The test report shall include a description of the tested hold-down device and/or assembly and device, including a drawing detailing all pertinent dimensions of the assembly and device. The description shall also include information concerning each component of the tested hold-down assembly.

11.2 The test report shall include the measured steel mechanical properties of the hold-down device, and cold-formed steel member(s).

11.3 The test report shall include a description of any modifications made to the cold-formed steel member(s) used in hold-down assembly testing.

11.4 The test report shall include a description of the bolts, screws, welds or other fasteners, and the anchor bolt/rod length dimension as shown in Figures 2(a), 2(b), 3(a), and 3(b).

11.5 When testing a bolted hold-down device, it shall be reported if the bolt threads are excluded or included in the shear plane between the device and the steel fixture.

11.6 The test report shall include a detailed drawing of the test setup, depicting location and direction of load application, location of displacement instrumentation and their point of reference, and details of any deviations from the test requirements as stipulated in Sections 6, 8, and 9. Additionally, photographs shall supplement the detailed drawings of the test setup.

11.7 The test report shall include individual load-versus-deformation values and curves, as plotted directly, or as reprinted from data acquisition systems.

11.8 The test report shall include individual load values observed, description of the nature, type and location of failure exhibited by each hold-down assembly or device tested, and a description of the general behavior of the test assembly or device during load application. Additionally, photographs shall supplement the description of the failure mode(s).

11.9 The test report shall include a description of the test method and loading procedure used, rate of loading or rate of motion of the crosshead movement.

AISI S914-08

Test Standard for Joist Connectors Attached to Cold-Formed Steel Structural Framing

1. Scope

1.1 This Standard provides a method to determine both the strength and deformation behavior of joist connectors used in cold-formed steel light-frame construction.

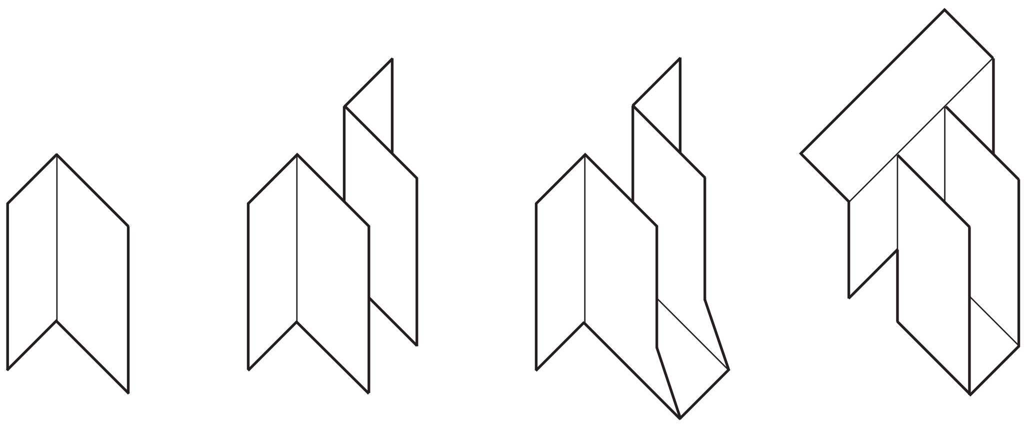

Figure 1: Typical Joist Connectors

User Note:
Illustrated in Figure 1 are some typical joist connectors, but there are many other configurations.

1.2 This Standard applies where the primary action of the joist is to impose a shear reaction to the joist connector. This Standard does not apply where a primary action of the joist is to impose an axial, bending or torsional reaction to the joist connector.

User Note:
If unrestrained by bracing, the asymmetry of typical c-shape joists would cause a torsional reaction.

A joist that complies with the continuously braced design provisions of AISI S210 would satisfy the requirements of Section 1.2.

1.3 This Standard applies to joist connectors attached to the cold-formed steel structural framing by use of welds or fasteners.

1.4 This Standard consists of Sections 1 through 11 inclusive.

2. Referenced Documents

The following documents or portions thereof are referenced within this Standard and shall be considered as part of the requirements of this document.

a. American Iron and Steel Institute (AISI), Washington, DC:

S100-07, North American Specification for the Design of Cold-Formed Steel Structural Members

S210-07, North American Standard for Cold-Formed Steel Framing – Floor and Roof System Design

b. ASTM International (ASTM), West Conshohocken, PA:

A370-07b, Standard Test Methods and Definitions for Mechanical Testing of Steel Products

E6-07b, Standard Terminology Relating to Methods of Mechanical Testing

IEEE/ASTM-SI-10-02, American National Standard for Use of the International System of Units (SI): The Modern Metric System

3. Terminology

Where the following terms appear in this Standard they shall have the meaning as defined herein. Terms not defined in Section 3 of this Standard, AISI S100 or ASTM E6 shall have the ordinary accepted meaning for the context for which they are intended.

Connection. Combination of structural elements and joints used to transmit forces between two or more members.

Connector. Device used to transmit forces between cold-formed steel structural members and other structural elements.

Fastener. Bolts, screws, power-driven pins or nails, clinches, or other mechanical devices.

Joist. Structural member primarily used in floor and ceiling framing.

Joist Connector. Connector, such as a clip angle(s) or joist hanger used to transmit forces between a *joist* and its support.

Joist Hanger. Joist connector with a seat.

4. Units of Symbols and Terms

Any compatible system of measurement units shall be permitted to be used in this Standard, except where explicitly stated otherwise. The unit systems considered in this Standard shall include U.S. customary units (force in kips and length in inches), and SI units (force in Newtons and length in millimeters) in accordance with IEEE/ASTM-SI-10.

5. Precision

5.1 Loads shall be recorded to a precision of 1 percent of the anticipated ultimate load during application of test loads.

5.2 Deflections shall be recorded to a precision of 0.001 in. (0.025 mm).

6. Test Fixture

The test fixture shall consist of either:

(a) A hydraulic or screw operated testing machine capable of operating at a constant rate of motion of the movable crosshead or a constant rate of loading, and a calibrated force-measuring device, or

(b) A hydraulic cylinder with a steel fixture, and a calibrated load cell.

User Note:
It is recommended that ASTM E4-07, Standard Practices for Force Verification of Testing Machines be used as applicable.

7. Test Specimen

7.1 The test specimen shall consist of the tested joist connectors and the fasteners used to connect the joist connectors to the joists and to the supporting members.

7.2 The number of specimens tested shall comply with the requirements of Section F1 of AISI S100.

7.3 The steel properties of the tested joist connectors, including yield stress, tensile strength, percent elongation, and uncoated base steel thickness shall be determined. Standard tensile tests of the steel from which the joist connectors were produced shall be conducted in accordance with ASTM A370 and Section F1.1(c) of AISI S100.

7.4 Fasteners used in joist connector testing shall be selected at random from one manufacturer's lot and installed in a manner that is representative of field conditions.

7.5 Welding, clinching or other fastening techniques shall be permitted in a manner that is representative of field conditions.

8. Test Setup

8.1 The test setup shall consist of cold-formed steel joist(s) and two supporting members representative of field conditions, and the joist connectors and fasteners to be evaluated (See Figure 2).

8.2 Supporting members shall be long enough to provide the intended contact surface for the joist connector; e.g., space for fasteners and bearing as applicable.

8.3 To avoid an unintentional load path, joist lengths shall be long enough to prevent contact between joist connectors and any material other than the attached supporting members and joist(s). A minimum horizontal clear distance (H) of 3 in. (76 mm) or 1/3 the joist depth, whichever is smaller, shall be provided between the load transfer block and the nearest portion of the joist connector, such as the outstanding leg of a clip angle or the seat of a joist hanger, as applicable.

8.4 The deflection device(s) shall measure the relative vertical movement between the end of the joist(s) and the supporting member. The deflection shall be measured no further than 1½ in. (38 mm) from the end of the joist (top, bottom, or side). The deflection device shall be placed within 1 in. (25 mm) from the end of the joist where the joist setup is inverted for uplift testing.

8.5 To avoid friction between the joist and supporting member, a minimum gap of 1/8 in. (3.2 mm) shall be provided between the end of each joist and abutting material, such as the adjacent supporting member or any portion of the joist connector that is perpendicular to the end of the joist. Prior to loading, the 1/8 in. (3.2 mm) gap shall be maintained by providing shims, or equivalent means.

8.6 To minimize load transfer due to friction, a low friction material, such as Teflon® or polyethylene, with a thickness no greater than 0.063 in. (1.6 mm) shall be inserted into the 1/8 in. (3.2 mm) gap prior to loading.

8.7 To avoid an unintentional load path, the joist bottom flange and seat of the joist hanger, as applicable, shall be prevented from having direct contact with the supporting member during the test.

8.8 To avoid an unintentional load path, no portion of the joist connector shall bear on any support other than the supporting member during the test. This shall be accomplished by either:

(a) Using raised supports with a minimum supporting member overhang of 1/8 in. (3.2 mm) at the inside edges, or

(b) Using supporting members that are deeper than the joist by an amount sufficient to ensure that neither the joist nor the joist connector contact the test bed.

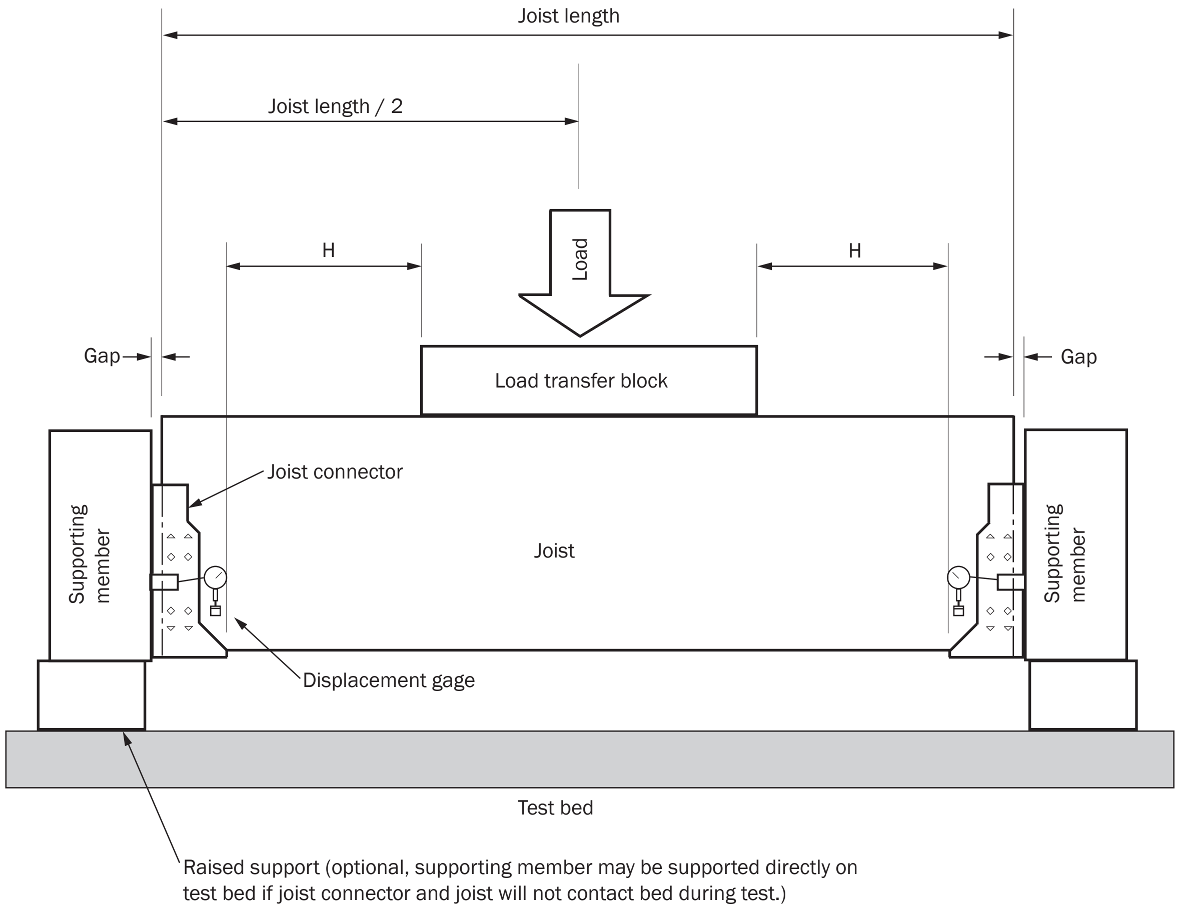

Figure 2: Test Set-up for Joist Connector

8.9 Reinforcement of joist members at the area of load application shall be permitted to prevent member failure in bending, shear, or web crippling at the applied load to ensure a failure of the joist connector, fasteners or supporting member, or of the joist due to bearing at the joist connector. The length of joist reinforcement shall be no closer than 2 in. (51 mm) from the end of each joist connector.

8.10 To prevent rotation of the supporting members (See Figure 2) inward towards the joist, blocking between the supporting members or another bracing method shall be provided. Tension reinforcement between the supporting members shall be permitted to prevent rotation of the supporting members outward away from the joist. Such reinforcement shall not contact the joist connectors or otherwise interfere with their performance.

9. Test Procedure

9.1 An initial load, or preload, shall be permitted to be applied to seat the assembly. This preload shall not exceed 10 percent of the average ultimate load.

9.2 The specimen shall be loaded such that the load is applied with reference to the intended application of the joist connector. The test load shall be applied at a uniform rate between 0.03 and 0.10 in. (0.76 to 2.54 mm) per minute until failure or maximum load. Loads shall be recorded to a precision of ± 1 percent of the ultimate load during application of test loads.

9.3 Load-deflection characteristics of the joist connector shall be determined. Deflections shall be recorded at a sufficient number of load levels to permit the establishment of a load-deflection curve. At least eight readings shall be taken prior to reaching the deflection limit state. The deflection limit shall be 1/8 in. (3.2 mm), unless otherwise defined by the applicable building code or a design standard approved by the authority having jurisdiction.

10. Data Evaluation

10.1 Evaluation of the test results and the determination of the available strength (i.e., allowable strength and/or design strength [resistance]) shall be made in accordance with the procedures described in Section F1 of AISI S100.

10.2 No test result shall be eliminated unless a rationale for its exclusion can be given.

11. Test Report

11.1 The test report shall include a description of the test specimens, including a drawing detailing all pertinent dimensions.

11.2 The test report shall include the measured steel mechanical properties of the joist connectors and joists.

11.3 The test report shall include a description of any modifications made to the joists.

11.4 The test report shall include a description of the bolts, screws, welds or other fasteners.

11.5 The test report shall include a detailed drawing of the test setup, depicting location and direction of load application, location of displacement instrumentation and their point of reference, and details of any deviations from the test requirements as stipulated in Sections 6, 8, and 9. Additionally, photographs shall supplement the detailed drawings of the test setup.

11.6 The test report shall include individual load-versus-deformation values and curves, as plotted directly, or as reprinted from data acquisition systems.

11.7 The test report shall include individual and average maximum test load values observed, description of the nature, type and location of failure exhibited by each specimen tested, and a description of the general behavior of the test fixture during load application. Additionally, photographs shall supplement the description of the failure mode(s).

11.8 The test report shall include a description of the test method and loading procedure used, and rate of loading or rate of motion of the crosshead movement.

SECTION 2 - BIBLIOGRAPHY OF TEST PROCEDURES PERTINENT TO COLD-FORMED STEEL

The following list of U.S. and Canadian publications on testing is provided for the convenience of the Manual user. No representation of correctness or completeness is implied.

ASTM Publications:

Sheet Steel, Mechanical Testing, General

ASTM A370 Standard Test Methods and Definitions for Mechanical Testing of Steel Products

ASTM E6 Standard Terminology Relating to Methods of Mechanical Testing

Sheet Steel, Mechanical Testing, Calibration and Verification

ASTM E4 Standard Practices for Force Verification of Testing Machines

ASTM E74 Standard Practice of Calibration of Force-Measuring Instruments for Verifying the Force Indication of Testing Machines

ASTM E83 Standard Practice for Verification and Classification of Extensometer Systems

Sheet Steel, Mechanical Testing, Tension

ASTM E8 Standard Test Methods for Tension Testing of Metallic Materials

ASTM E21 Standard Test Methods for Elevated Temperature Tension Tests of Metallic Materials

Sheet Steel, Mechanical Testing, Compression

ASTM E9 Standard Test Methods of Compression Testing of Metallic Materials at Room Temperature

Sheet Steel, Chemistry

ASTM E350 Standard Test Methods for Chemical Analysis of Carbon Steel, Low-Alloy Steel, Silicon Electrical Steel, Ingot Iron, and Wrought Iron

Sheet Steel, Coating Tests

ASTM A90 Standard Test Method for Weight [Mass] of Coating on Iron and Steel Articles with Zinc or Zinc-Alloy Coatings

ASTM A1004 Standard Practice for Establishing Conformance to the Minimum Expected Corrosion Characteristics of Metallic, Painted-Metallic, and Nonmetallic-Coated Steel Sheet Intended for Use as Cold Formed Framing Members

ASTM E376 Standard Practice for Measuring Coating Thickness by Magnetic-Field or Eddy-Current (Electromagnetic) Examination Methods

ASTM E797 Standard Practice for Measuring Thickness by Manual Ultrasonic Pulse-Echo Contact Method

Sheet Steel, Forming Parameters

ASTM E517 Standard Test Method for Plastic Strain Ratio r for Sheet Metal

Structural Testing of Sheet Steel Assemblies

ASTM C645	Standard Specification for Nonstructural Steel Framing Members
ASTM C955	Standard Specification for Load-Bearing (Transverse and Axial) Steel Studs, Runners (Tracks), and Bracing or Bridging for Screw Application of Gypsum Panel Products and Metal Plaster Bases
ASTM E72	Standard Test Methods of Conducting Strength Tests of Panels for Building Construction
ASTM E73	Standard Practice for Static Load Testing of Truss Assemblies
ASTM E330	Standard Test Methods for Structural Performance of Exterior Windows, Doors, Skylights and Curtain Walls by Uniform Static Air Pressure Difference
ASTM E455	Standard Test Methods for Static Load Testing of Framed Floor or Roof Diaphragm Constructions for Buildings
ASTM E564	Standard Practice for Static Load Test for Shear Resistance of Framed Walls for Buildings
ASTM E575	Standard Practice for Reporting Data From Structural Tests of Building Constructions, Elements, Connections, and Assemblies
ASTM E695	Standard Method for Measuring Relative Resistance of Wall, Floor, and Roof Constructions to Impact Loading
ASTM E1592	Standard Test Method for Structural Performance of Sheet Metal Roof and Siding Systems by Uniform Static Air Pressure Difference

Acoustical Testing of Sheet Steel Assemblies

ASTM E90	Standard Test Method for Laboratory Measurement of Airborne Sound Transmission Loss of Building Partitions and Elements
ASTM E336	Standard Test Method for Measurement of Airborne Sound Attenuation between Rooms in Buildings
ASTM E413	Classification for Rating Sound Insulation
ASTM E492	Standard Test Method for Laboratory Measurement of Impact Sound Transmission Through Floor-Ceiling Assemblies Using the Tapping Machine

Moisture Testing of Sheet Steel Assemblies

ASTM E96	Standard Test Methods for Water Vapor Transmission of Materials
ASTM E331	Standard Test Method for Water Penetration of Exterior Windows, Skylights, Doors and Curtain Walls by Uniform Static Air Pressure Difference
ASTM E547	Standard Test Method for Water Penetration of Exterior Windows, Skylights, Doors and Curtain Walls by Cyclic Static Air Pressure Difference

Fire Testing of Sheet Steel Assemblies

ASTM E119	Standard Test Methods for Fire Tests of Building Construction and Materials

Welding Test Procedures

ASTM E390	Standard Reference Radiographs for Steel Fusion Welds

Fatigue Test Procedures

ASTM E466 Standard Practice for Conducting Force Controlled Constant Amplitude Axial Fatigue Tests of Metallic Materials

ASTM E467 Standard Practice for Verification of Constant Amplitude Dynamic Forces in an Axial Fatigue Testing System

ASTM E468 Standard Practice for Presentation of Constant Amplitude Fatigue Test Results for Metallic Materials

ASTM E739 Standard Practice for Statistical Analysis of Linear or Linearized Stress-Life (S-N) and Strain-Life (ε-N) Fatigue Data

Joining and Fastening Test Procedures

ASTM E488 Standard Test Methods for Strength of Anchors in Concrete and Masonry Elements

ASTM E767 Standard Test Method for Shear Strength Properties of Metal Connector Plates

General References

ASTM E631 Standard Terminology of Building Constructions

IEEE/ASTM SI 10 American National Standard for Use of the International System of Units (SI): The Modern Metric System

Other Publications:

Test Procedure for Shear Resistance of Small-Scale Framed Wall Specimens, in "Diaphragm Braced Members and Design of Wall Studs," *Journal of the Structural Division,* ASCE, January 1976.

Canadian Sheet Steel Building Institute, "Criteria for the Testing of Composite Slabs," CSSBI S2-02, March 2002.

SECTION 3 - EXAMPLE PROBLEM

EXAMPLE VI-1: Computing ϕ And Ω Factors From Test Data

Given:

1. An unusual weld configuration made up of a group of arc seam welds is tested giving the following test strengths.

Test	Strength (kips)
1.	5.60
2.	6.00
3.	5.80
4.	5.90

The failure mode is plate tearing for all tests.

Required:

1. Determine the resistance factor, ϕ, for this assembly.

2. Determine the factor of safety, Ω, for this assembly.

Solution:

1. Calculate the mean test value

$$R_n = (5.6 + 6.0 + 5.8 + 5.9)/4 = 5.83$$

2. Check maximum deviation

Test 1 controls by inspection.

$$(5.83 - 5.60)/5.83 = 0.039 < 0.15 \quad \text{OK}$$

3. Compute the correction factor, C_p

$$C_p = (1 + 1/n)m/(m - 2) \qquad (\textit{Eq. } \text{F1.1-3})$$

where

n = number of tests = 4

$m = n - 1 = 3$

$$C_p = (1 + 1/4)3/(3 - 2) = 3.75 \qquad (\textit{Eq. } \text{F1.1-3})$$

4. Compute the standard deviation of the test results

$$s = \sqrt{\frac{\sum_{i=1}^{n}(x_i - \bar{x})^2}{n-1}}$$

$$= \sqrt{\frac{(5.6-5.83)^2 + (6.0-5.83)^2 + (5.8-5.83)^2 + (5.9-5.83)^2}{4-1}}$$

$= 0.171$

5. Compute the coefficient of variation of the test results, V_p

$V_P = s/R_n$

$= 0.171/5.83$

$= 0.029 < 0.065 \therefore$ use 0.065

6. Obtain M_m, F_m, V_M, and V_F from Table F1 of the *Specification*

For arc seam welds - Plate Tearing

$M_m = 1.10$
$F_m = 1.00$
$V_M = 0.10$
$V_F = 0.10$

7. Determine P_m, β_o, V_Q and C_ϕ

$P_m = 1.0$ (always)
$\beta_o = 3.5$ (for connections for the United States)
$V_Q = 0.21$ (always)
$C_\phi = 1.52$ (for the United States)

8. Compute ϕ

$$\phi = C_\phi (M_m F_m P_m) e^{-\beta_o \sqrt{V_M{}^2 + V_F{}^2 + C_P V_P{}^2 + V_Q{}^2}} \quad (Eq.\ F1.1\text{-}2)$$

$$= 1.52\left[(1.10)(1.0)(1.0)\right] e^{-3.5\sqrt{0.10^2 + 0.10^2 + (3.75)0.065^2 + 0.21^2}}$$

$$= 0.62$$

9. Compute Ω

$$\Omega = \frac{1.6}{\phi} \quad (Eq.\ F1.2\text{-}2)$$

$$= \frac{1.6}{0.62}$$

$$= 2.6$$

American
Iron and Steel
Institute

1140 Connecticut Avenue NW
Suite 705
Washington DC 20036
www.steel.org

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